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PREDICTION OF PAVEMENT ROUGHNESS.(U)
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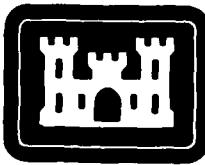
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PREDICTION OF PAVEMENT ROUGHNESS

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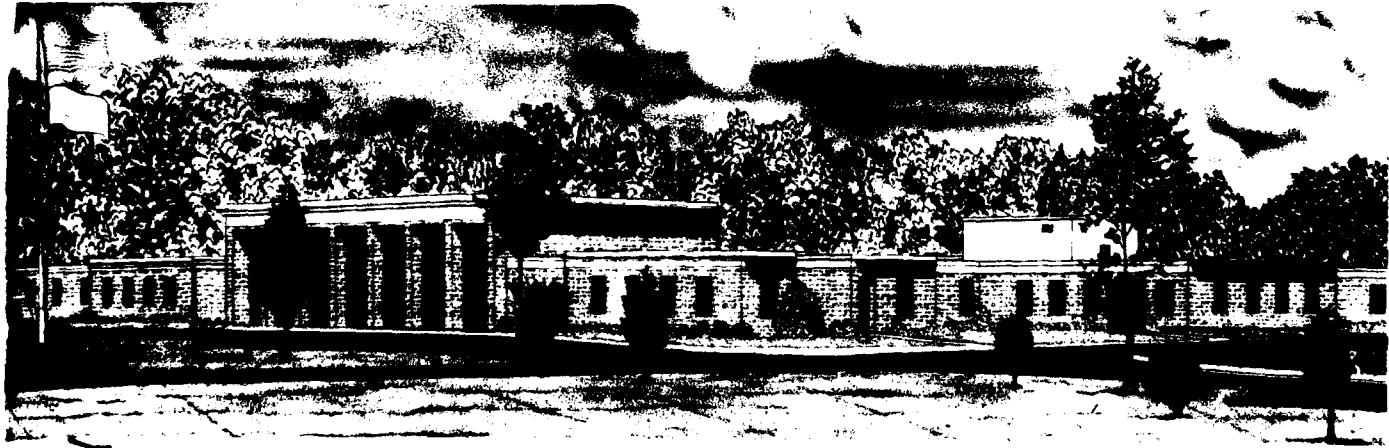
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P. O. Box 631, Vicksburg, Miss. 39180

September 1982
Final Report

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Department of the Army
Washington, D. C. 20315

Under Project No. 4A161101A910, Task Area 02
Work Unit 13906

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) A simplified procedure has been developed for considering roughness in pavement design. The procedure utilizes statistical parameters to generate stochastic pavement material properties and profiles. A methodology was developed for predicting the rutting of a pavement section due to applied traffic. Thus the predicted rut depth can be applied to the profiles yielding a predicted surface profile. This profile can then be used to determine a measure of pavement roughness.			

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Preface

The study reported herein was performed during the period January 1980 to September 1982 under Department of the Army Project No. 4A161I01A91D, In-House Laboratory Independent Research (ILIR) Program, Task Area A2, Work Unit 13906, sponsored by the Assistant Secretary of the Army (R&D).

The report was prepared by Dr. W. R. Barker of the Pavement Systems Division (PSD), Geotechnical Laboratory (GL), U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss. During the conduct of the study Mr. A. H. Joseph was Chief of PSD and Messrs. H. H. Ulery and J. W. Hall were Acting Chiefs of PSD. The Chief of GL was Dr. W. F. Marcuson III.

Commanders and Directors of the WES during the performance of this work and the preparation of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. Fred R. Brown

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Conversion Factors, U. S. Customary to Metric (SI)
Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (force) per square inch	6.894757	megapascals
knots (international)	0.5144444	metres per second
pounds (force) per square inch	6894.757	pascals

PREDICTION OF PAVEMENT ROUGHNESS

Introduction

For many years the emphasis in pavement research has been directed toward the development of a mechanistic approach to pavement design. Nearly all such design procedures have been based on an empirical correlation between pavement distress and computed pavement response parameters. The computed response parameters are normally based on a deterministic evaluation of material properties and pavement section geometrics. These design procedures do represent an advancement in the state of the art of pavement design in that they provide a mechanism for considering basic material properties and pavement response. A shortcoming of these procedures is the fact the procedures do not lead directly to a prediction of the functional performance of a pavement. For military airfields the concept of functional requirements is particularly important. The importances of functional requirements are illustrated in the studies of methods for repairing bomb damage and the minimum pavement requirement for alternate pavements to be used only in emergency. In each of these examples the conventional definition of pavement distress becomes meaningless. The principal consideration is the effect the pavement surface has on the operating aircraft. An example of the acceleration forces generated by a B-52 aircraft operating on a hypothetical sinusoidal profile is shown in Fig. 1 (Horn 1977). For the hypothetical sinusoidal profile the critical wave length for various aircraft speeds is shown in Fig. 2 (Horn 1977). Thus it is seen that the forces generated are dependent on both the amplitude, frequency of rutting, and speed of the aircraft. With this in mind then the prediction of pavement roughness is seen to involve not only the prediction of a rut depth but also the prediction of the distribution of the rutting.

Scope

This report presents a general methodology for predicting pavement roughness and a specific methodology for predicting rutting in flexible pavements. No specific method was developed for predicting of roughness for rigid pavements nor for the roughness caused by swell and frost heave of pavements.

Approach

The prediction of pavement roughness involves the generation of hypothetical pavements that meet known statistical parameters for surface profile, thickness and material properties. For each of these pavements a mechanistic model is used to predict the development and distribution of rutting with traffic. Thus at any time in traffic a profile for each pavement can be generated and the resulting profile analyzed as to the effect on using aircraft by using an aircraft simulation program, such as the one described by Horn (1977). If a sufficient number of hypothetical pavements are analyzed, then a distribution of roughness parameters can be generated such that the probability that a certain roughness develops is predicted.

Generation of Hypothetical Pavements

The generation of a series of parametric values that meet statistical data may be accomplished in a number of ways. One of the simplest is the use of a distribution function by which a series of values will be distributed. The values can be selected randomly from the distribution such that resulting set will have no order but will have a specified mean and standard deviation. The most commonly used function, the normal distribution, has some undesirable properties, mainly that the possible values are unbounded and the data are symmetrically distributed. Possibly a better distribution function would be the Beta function as described by Harr (1977). The Beta function overcomes some of the disadvantages of the normal distribution in that it may be skewed and has a specified minimum and maximum value. A computer program was written (Appendix A) that provides a set of values that are distributed according to the Beta function and has a given mean, standard deviation, minimum value and a maximum value.

Consider for example the generation of 40 values having a mean of 6.7, a standard deviation of 2.2, a minimum value of 2 and a maximum value of 15. The values generated by the computer program having the required statistical properties are given in Appendix A. The distribution of the values is shown in Fig. 3. Such a distribution could very well represent the values of soil strength in terms of California Bearing Ratio (CBR) where CBR values are bounded and have a skewed distribution. The parameters describing a pavement section can also be generated using the Beta distribution function.

From a statistical point the randomly generated values satisfy the requirement of the problem. If in the generation of the properties for a hypothetical pavement, the points for which properties are being generated are sufficiently far apart that the properties are independent of one another, then the procedure is correct. For airfield pavements where the critical wavelength is rather long and properties are only generated for these points, then the routine may be sufficiently correct to use. If the shorter wavelengths are to be considered and the properties at one point are influenced by the properties of adjacent points, then additional restraints must be placed on the generation scheme.

Dr. Per Ullidtz developed such a routine for prediction of roughness in highway pavements where the critical wavelengths are much shorter than in airfield pavements. Dr. Ullidtz first tried what he called a random walk in which the mean of a parameter distribution was set equal to the value of the previous point. This procedure resulted in unrealistically smooth pavements and thus the scheme was modified to what Dr. Ullidtz named the modified random walk. This scheme is different in that the mean of the parameter distribution is a projection of the values of the previously determined two values. The two schemes are illustrated in Fig. 4 (Fig. 2 of Ullidtz).

In a study of the significant characteristics of runway roughness, Berens and Newman (Berens, 1973, AFFDL-TR-73-109) developed a computer program for generating hypothetical runway profiles that had a specified power spectral density (PSD).

also the PSD of a number of different pavements were determined and a typical new pavement PSD was estimated. Thus using the computer program, a profile could be determined that would simulate a new pavement. In the simulation process a methodology was developed for random spacing of the different waves along the pavement profile. An example of the results of the simulation process is shown in Fig. 5.

The problem of developing probabilistic information about the engineering parameters of a space also exists in other areas of geotechnical engineering. Consider the paper by Wu and Wong (Wu and Wong 1981) that describes the case history of a problem in probabilistic soil exploration. In this case, the soil properties are measured at several locations. The probability

contours are developed based on a proximity rule. For pavement design, it is usually not possible to have measured parameters since the pavement system is to be constructed in the future. Still such a proximity rule could be used particularly if short wavelengths are important. The basic concept would be to randomly select parameters at distances such that they can be considered mutually independent. These values can then be considered the same as measured values and a proximity rule used to develop parameters between the randomly determined values.

Prediction of Pavement Rutting

General

Two basic approaches were available for prediction of the pavement rutting - the statistical and mechanistic. Barber et al. (1978) conducted a study of numerous pavements and developed a statistical model for prediction of the rut depth as a function of CBR, thickness, and traffic. This particular model provides the rut depth as a deterministic value or a statistical value. If the rut depth is determined statistically, i.e. in terms of a mean and variance, then the rut depth can be distributed directly. Although Barber's model currently exists, for reasons of versatility and laboratory testing considerations a mechanistic model for prediction of rutting was developed.

Permanent Strain Model

The mechanistic model is based on a laboratory-determined relationship between the permanent deformation (ϵ_p) and the resilient deformation (ϵ_R) and state-of-stress along a vertical axis of the pavement system. The first work accomplished toward developing the mechanistic model was to review available laboratory data for the development of the relationship between ϵ_p and ϵ_R and to show the feasibility of using the approach for predicting rut depth. This study was rather lengthy and involved; therefore the results are presented separately in Appendix B. Although the methodology presented in Appendix B was a viable methodology, it did not lend itself to a computerized procedure.

From the data analyzed in Appendix B it was apparent that the relationship between ϵ_p and ϵ_R is influenced by the soil strength. To develop the relationship with soil strength the WES data were plotted as shown in Fig. 6. For each of the repetition levels a relationship is shown between the ratio of ϵ_p and ϵ_R and the resilient modulus (M_R). By considering the relationship shown in Fig. 6,

the relationship that

$$\frac{\epsilon_p}{\epsilon_R} = .14 \left(\frac{70,000}{M_R} \right)^R$$

where

$$R = 0.4 (\text{Stress Reps})^{0.12}$$

$$M_R = \frac{\epsilon_d}{\epsilon_R}$$

σ_d = repeated deviator stress in laboratory triaxial test

ϵ_R = measured resilient strain in laboratory triaxial test

is developed.

As an independent check of the relationship, a comparison was made with experimental data reported by Ogawa (1972). This comparison is shown in Fig. 7.

Model for Resilient Modulus

For the purpose of this study a comparison with actual test data would be most desirable. One problem is that the strain model is based on M_R where the measured material property in the field test sections is the CBR. Making the conversion from CBR to M_R is rather difficult since the M_R is a function of stress as well as material properties. This conversion was made by developing an empirical correlation between field CBR and M_R . The correlation may be expressed by the equation

$$M_R = 10^x$$

where

$$x = 4.5682 - \frac{1.9661}{8.55 \text{ CBR}} (6.5 + \sigma_d)$$

σ_d = repeated deviator stress

The relationship is shown graphically in Fig. 8.

This correlation was developed from data obtained in connection with a study (Parker et al. 1979) to develop a correlation between plate bearing value and M_R . For the study a number of field sites were selected based on an attempt at having sites covering a wide range of soil types. At each site plate bearing and CBR tests were conducted and undisturbed soil samples were taken. Resilient modulus tests were conducted by the WES Geotechnical Laboratory. An example of the

laboratory resilient modulus data and comparison with the model is given in Fig. 9. With the models now it is possible, provided the stress distribution is known, to make predictions of permanent deformation for various prototype pavement tests that have been conducted at WES.

Stress Model

When considering the stress distribution in flexible pavements, the structural layers above the subgrade are divided into bound and unbound materials. Bound materials are capable of sustaining tensile stress and thus tend to distribute load as a slab; however, complete slab action is unlikely. Shrinkage and load associated cracking reduce the load-distributing characteristics of stabilized layers in pavement systems. In another study (the report to be published later) it was shown from data obtained in prototype testing of air-field pavements that bound layers give a load-distributing effect equivalent to a two-layered system having a modulus ratio (modulus of elasticity of the top layer divided by the modulus of elasticity of the bottom layer) between 3 and 4. This ratio will depend somewhat on the type material being stabilized and the amount of stabilization. For gravels that are well stabilized the ratio will approach 4; whereas for clays the ratio will be closer to 3. If there is very dense cracking in the stabilized material, the ratio could be lower but this is not likely to occur before complete failure of the pavement system.

Unbound structural layers, both crushed and uncrushed gravels, in a pavement system distributed the load equivalent to a two-layered system having a modular ratio of 1; that is to say the stresses will be distributed according to the Boussinesq stress model.

Since the procedure for predicting roughness requires the computation of stress many times, it is desirable to have a rapid means for performing these computations. For this a computer program was written using tabulated stress factors that are a function of the r/t ratio, r/z ratio, and the modulus ratio where r is the radius of the load area, t is the thickness of the structural layer, and z is depth to the point for computation of stress. An example of the stress distribution for the G-SA is given in Fig. 10.

Rutting Model

The rutting model consists of the permanent strain model and the stress distribution model. This model is illustrated in Fig. 11. There are some

assumptions that must be included with the use of the model. The first assumption is that a stress repetition is synonymous with a coverage. The second assumption is that no permanent strain occurs in bound layers. For cement or lime-stabilized material, this has been found to essentially be true as long as the bound materials completely fail. For asphalt bound materials rutting can occur within these layers. This is particularly true if a poor design is used in the mix. This report presents no rutting model for use in predicting rutting in the asphalt bound materials. Fortunately most military airfields have well-designed asphalt bound layers that are fairly thin and thus the rutting within these layers will be negligible. The third assumption is that although the permanent strain model was developed based on testing of subgrade soils, it is assumed to be useable for granular materials. This assumption can be justified from the data presented in Appendix B.

The rutting model as presented in Fig. 11 provides a deterministic evaluation of the rut depth. If the model described for generation of a hypothetical pavement is used for generation of the material properties and structural layer thicknesses and a sufficient number of sections are generated, then the variations in rut depth can be simulated. This procedure, referred to as the Monte Carlo procedure, can be used to determine the probability of the rut exceeding a certain value or, if used in connection with an aircraft simulation program, the probability of the acceleration forces exceeding a certain value.

Prof. M. Harr introduced in a short course a procedure he referred to as Rosenblueth's procedure that could be used to compute the rut depth in statistical terms. The procedure uses a finite difference procedure where the dependent value of a function is computed at both the mean plus a standard deviation and the mean minus a standard deviation of the independent variable. These two values are used to compute the mean and deviation of the dependent variable. The procedure for a single independent variable is for

$$Y = f(X)$$

then

$$\bar{Y} = 1/2 (X_+ + X_-)$$

and

$$S_y = 1/2 (X_+ - X_-)$$

where

\bar{Y} = mean of Y

S_y = standard deviation of Y

Y_+ = $F(\bar{X} + S_x)$

Y_- = $F(\bar{X} - S_x)$

\bar{X} = mean of X

S_x = standard deviation of X

The variance, v_y , of Y can be computed by

$$v_y = \frac{s_y^2}{\bar{Y}} \quad (100)$$

This procedure can be extended to multiple independent variables by considering the functional value for values of the mean plus or minus a standard deviation for each of the independent variables, i.e.,

$$\tilde{\mu}_1 = \frac{1}{2} (x_+^1 + x_-^1)$$

$$\tilde{s}_1 = \frac{1}{2} (x_+^1 - x_-^1)$$

$$\tilde{\mu}_2 = \frac{1}{2} (x_+^2 + x_-^2)$$

$$\tilde{s}_2 = \frac{1}{2} (x_+^2 - x_-^2)$$

where

x_+^1 = the mean plus a deviation for variable 1

x_-^1 = the mean minus a deviation for variable 1

x_+^2 = the mean plus a deviation for variable 2

x_-^2 = the mean minus a deviation for variable 2

$\tilde{\mu}$ = a mean of the function with respect to variable 1 with all other variables at the mean

\tilde{s}_1 = a standard deviation with respect to variable 2 with all other variables at the mean

$\tilde{\mu}_2$ = same as $\tilde{\mu}_1$ but with respect to variable 2

\tilde{s}_2 = same as \tilde{s}_1 but with respect to variable 2

Also the value of the function at the mean of all variables is used and is computed by

$$u^* = \frac{1}{2} (x^1, x^2)$$

where

x^1 = the mean of variable 1

x^2 = the mean of variable 2

Now the equation

$$\frac{u}{u^*} = \frac{\tilde{u}_1}{\tilde{u}_2} \cdot \frac{\tilde{u}_2}{u^*}$$

can be used to compute the mean, u , of the function. For variance the equations

$$\tilde{v}_1 = \frac{s_1}{\tilde{u}_1}$$

$$\tilde{v}_2 = \frac{s_2}{\tilde{u}_2}$$

are used to compute the variance, \tilde{v}_1 and \tilde{v}_2 , with respect to each of the variables.

Now the equation

$$1 - v^2 = (1 + \tilde{v}_1^2) (1 + \tilde{v}_2^2)$$

can be used to compute the variance of the function with respect to all variables.

The Rosenblueth procedure was combined with the rutting model and computerized to give a computer program for computing the rut depth in statistical terms.

The listing of this program is given in Appendix C. With the rut depth computed in statistical terms, the rut could be distributed directly to produce the runway profile.

Other Considerations

A methodology has been presented for generating a new airfield pavement profile and material properties that meet given statistical data. For flexible airfield pavements these data can be used to predict the development of permanent deformation with traffic and thus construct a prediction of the profile with traffic. Using the profile as input in an aircraft simulation program, a measure of roughness is obtained. The methodology developed thus far is rather crude but does illustrate the feasibility of the approach and provides basic models necessary for future development. As mentioned earlier, a pressing need is for the development of the proximity rules to be used in generating the initial pavement sections. Verification of the proximity rules should be accomplished by comparison with actual measured airfield data.

Also a factor that could be considered is the development of structural cracking. Models are currently available for both rigid and flexible pavements for the prediction of structural cracking, but the effect of the cracking on the roughness of a pavement is a relatively unknown quantity.

Due to the interaction of the aircraft and runway surface the loading to the pavement is not necessarily equal to the stated loading of the aircraft. The consideration of dynamic load in the procedure would add an order of magnitude in the solution process. Such loadings are a function of both profile and speed, and thus consideration of the determination of the dynamic loading would require continuous updating as the profile changed.

The profile could also be affected by climatic conditions, particularly in frost areas. For pavements that are not designed for full frost protection the frost heave is likely to be very critical. The weakening of the subgrade during spring thaw is another factor that should be considered as the permanent deformation that occurs during this period of time may be greater than the deformation occurring during the remainder of the year.

For rigid pavements the joints pose a special problem. There is some statistical chance of dowel failures, spalling, pumping, or other joint problems that could create a rough pavement. For rigid pavements, more so than flexible pavements, research aimed at solving the problems of predicting pavement roughness is almost nonexistent.

Summary

A simplified procedure has been developed for considering roughness in pavement design. The procedure used the variation in pavement section to generate a stochastic airfield pavement. A model was developed for predicting the permanent surface deformation with aircraft traffic. Thus the predicted deformation can be added to the initial profile to yield a pavement surface at any point in time. The predicted pavement section can be tested for roughness by the use of an aircraft simulation computer program.

Extension of this simplified procedure to provide consideration of some of the other factors that affect pavement roughness would require a large research effort. Although the research effort for complete development of the methodology is large, it is a project that should be undertaken. The return on such an effort in terms of better pavement design and pavement management data would justify the monies spent.

References

- Barber, V. C., Odom, E. C., and Patrick, R. W. 1978. "The Deterioration and Reliability of Pavements," Technical Report S-78-8, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
- Berens, A. P. and Newman, R. K. 1973. Technical Report No. AFFDL-TR-73-109, Air Force Flight Dynamics Laboratory, Air Force Systems Command, Wright-Patterson Air Force Base, Ohio.
- Harr, M. E. 1977. "Mechanics of Particulate Media," McGraw-Hill, New York
- Horn, W. J. 1977. "Airfield Pavement Smoothness Requirements," Technical Report S-77-12, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
- Ogawa, N. 1972. "The Permanent Deformation Characteristics of Compacted Sub-grade Soil Subjected to Repeated Loading," CE 299 Report, Transportation Division, Civil Engineering Department, University of California, Berkeley.
- Parker, F., Jr., Barker, W. R., Gunkel, R. C., and Odom, E. C. 1979. "Development of a Structural Design Procedure for Rigid Airport Pavements," Technical Report No. GL-79-4, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
- Ullidtz, P. 1978. "Computer Simulation of Pavement Performance," Report No. 18, The Institute of Roads, Transportation and Town Planning, The Technical University of Denmark, Lyngby, Denmark.
- Wu, T. H., and Wong, K. 1981. "Probabilistic Soil Exploration Case History," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 107, No. GT12, p 1693.

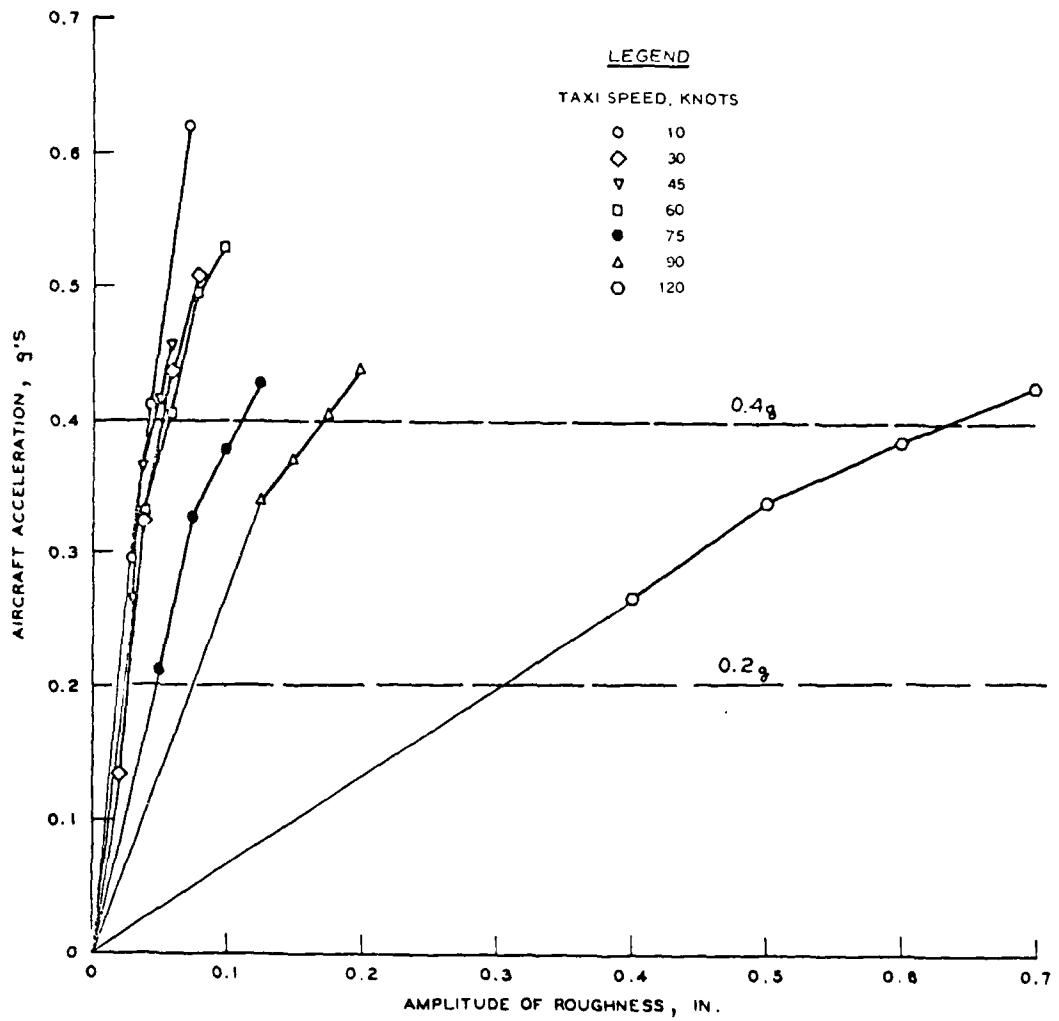


Figure 1. Acceleration forces for hypothetical sinusoidal profile; B-52H (after Horn 1977)

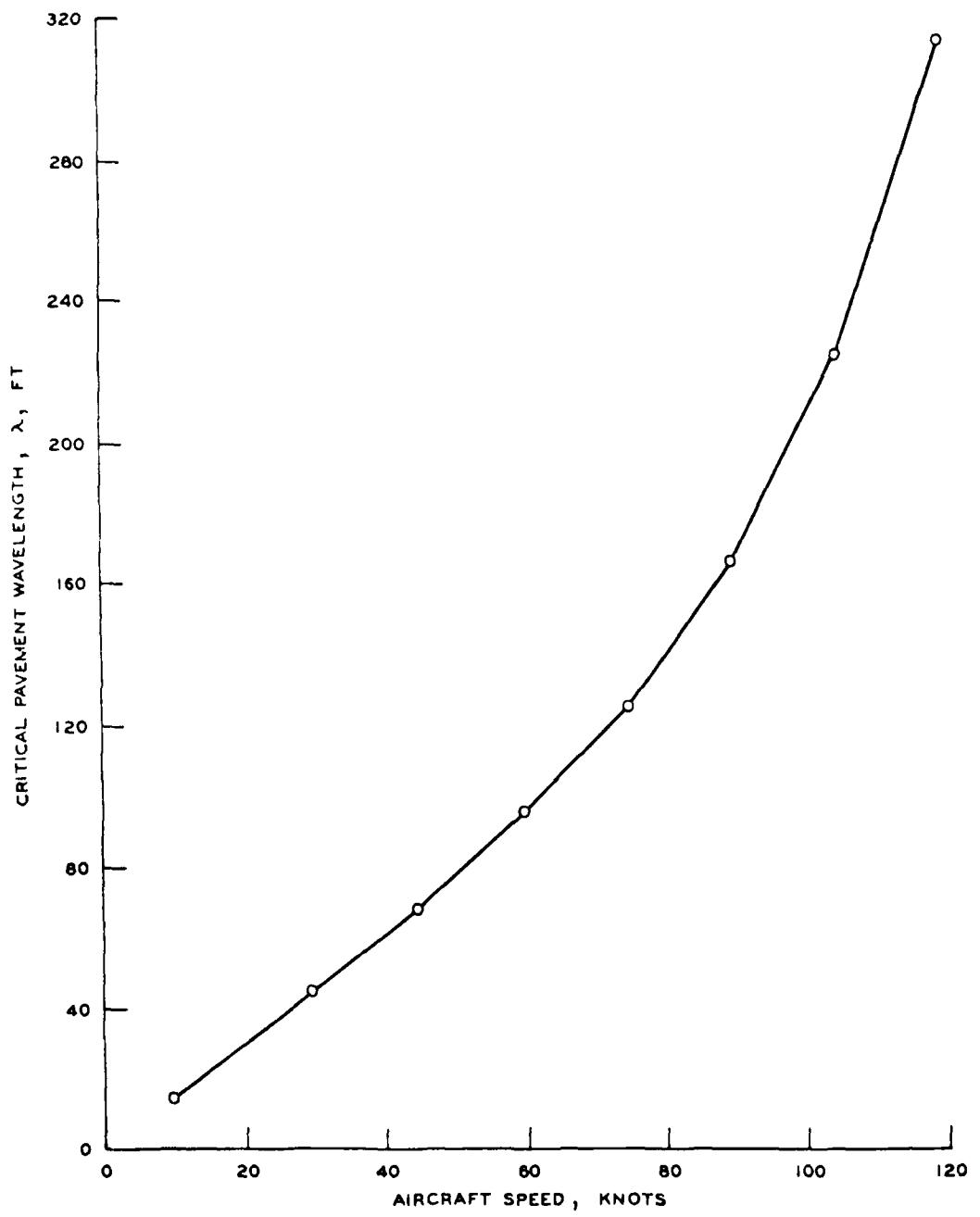


Figure 2. Critical pavement wavelength as a function of aircraft speed (after Horn 1977)

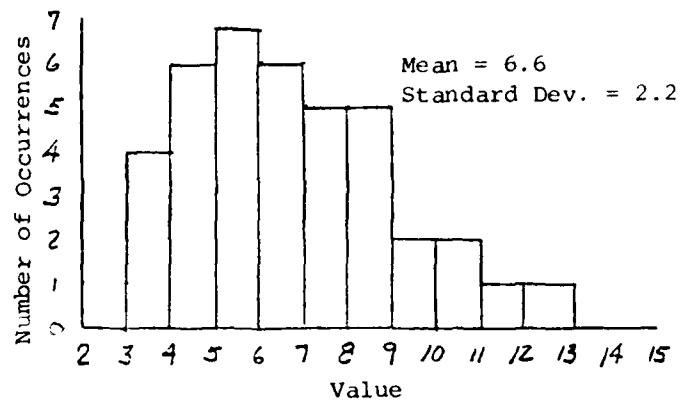


Figure 3. Histogram of values for Beta distribution

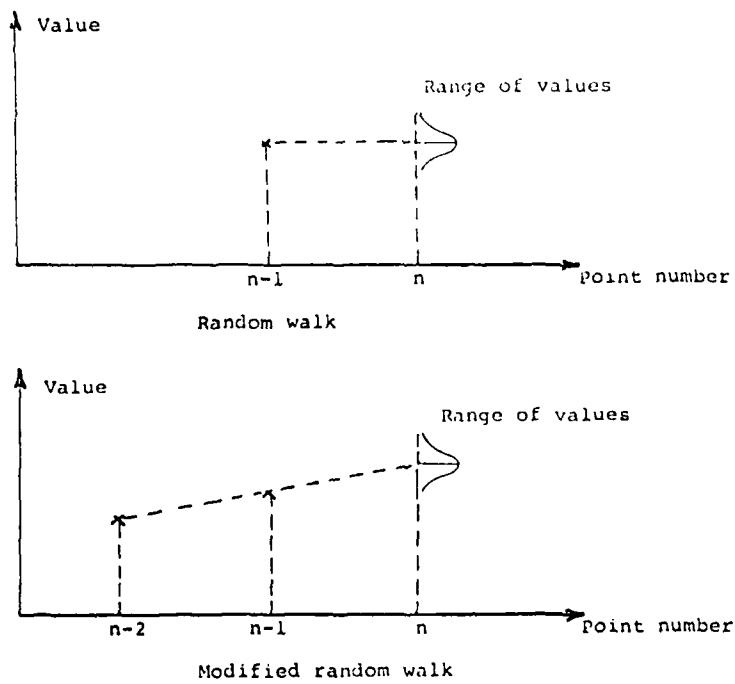


Figure 4. Illustration of random walk and modified random walk (after Ullidtz 1978)

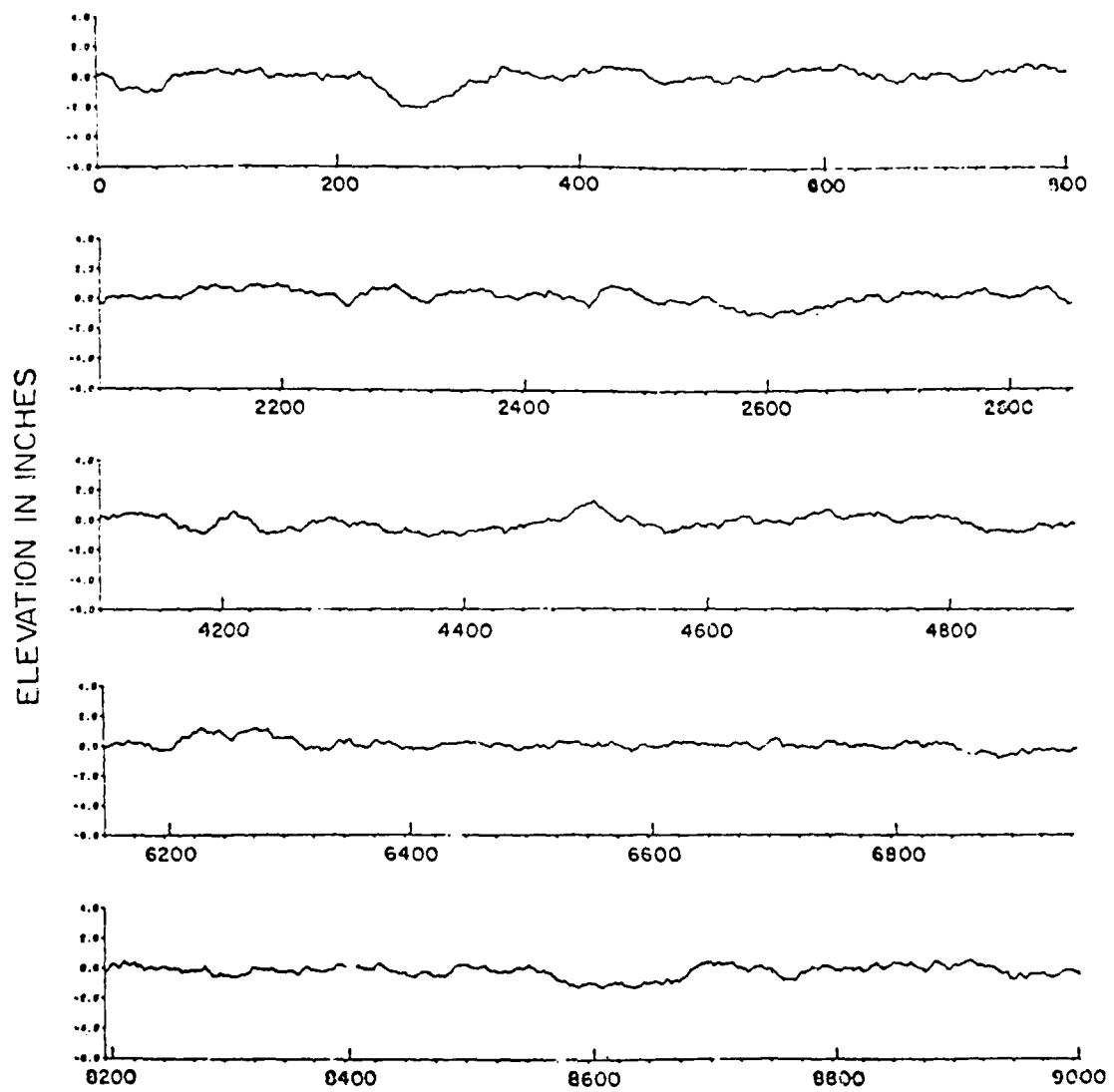
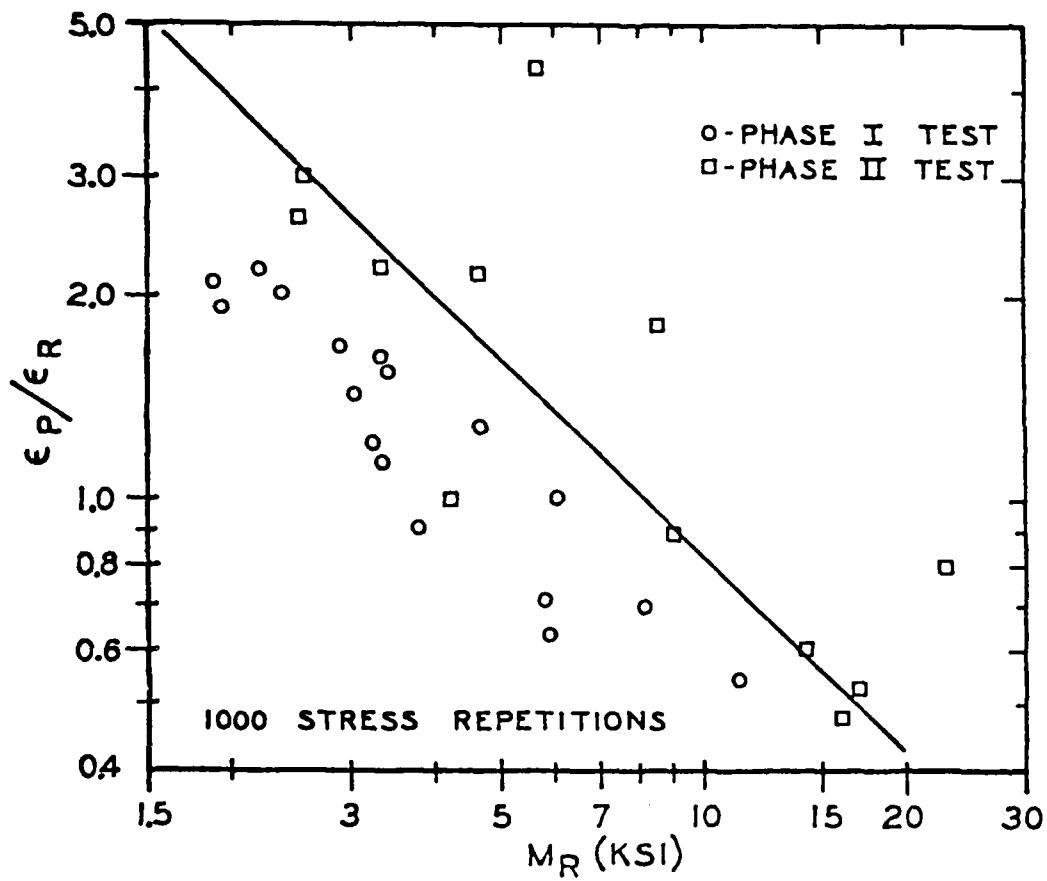
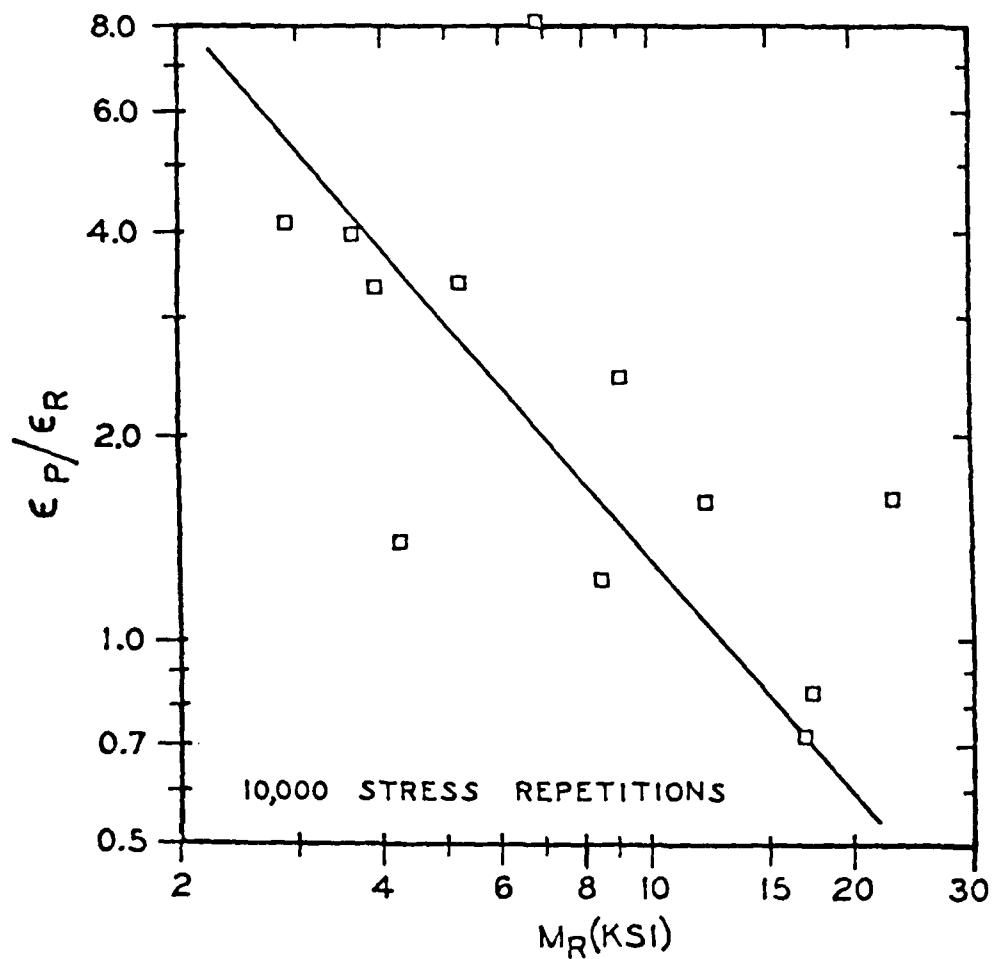


Figure 5. Illustration of generation of runway profile using power spectral density (after Berens and Newman 1973)



a. 1,000 stress repetitions

Figure 6. Relationship between ϵ_p/ϵ_R and M_R (Sheet 1 of 3)



b. 10,000 stress repetitions

Figure 6. (Sheet 2 of 3)

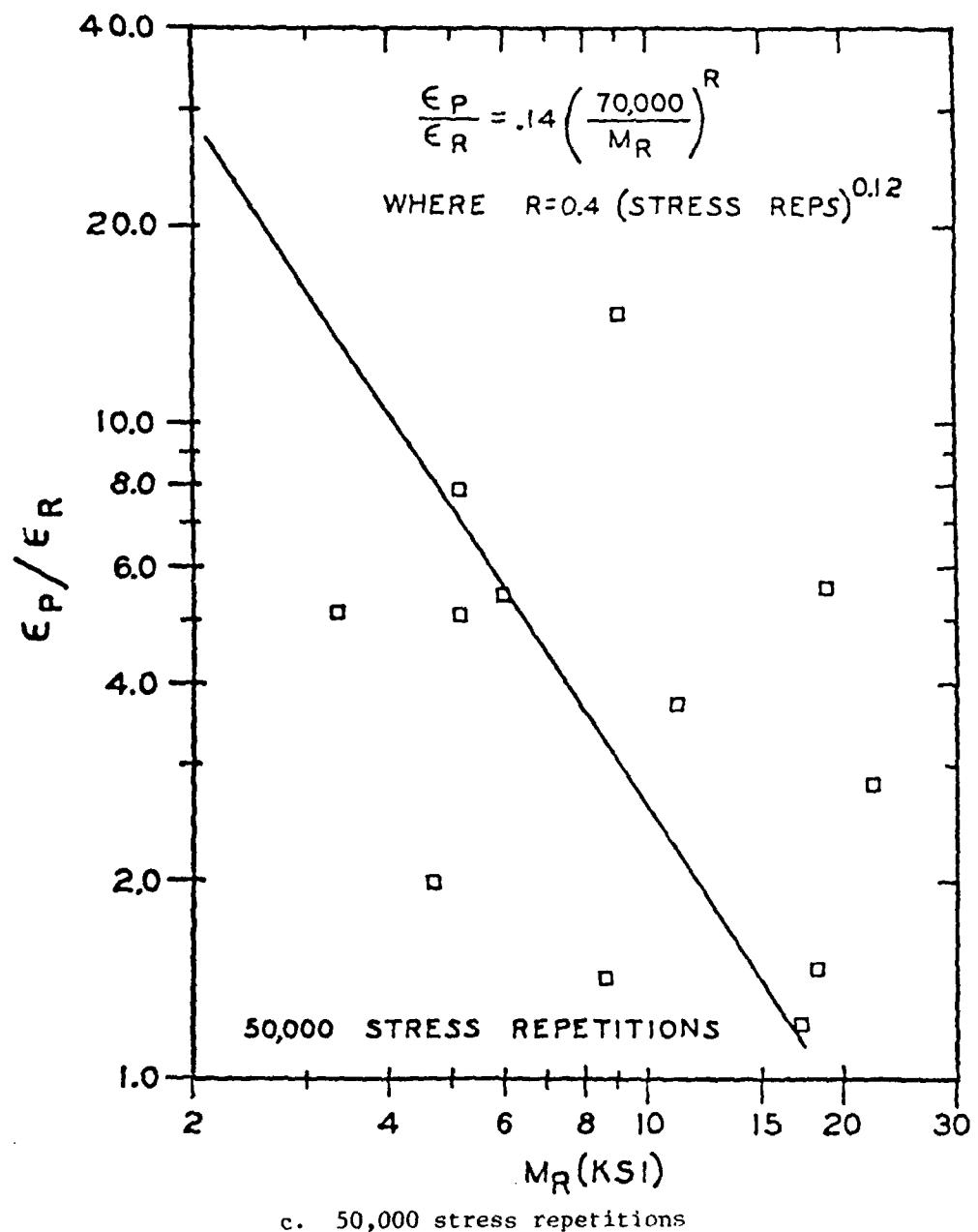


Figure 6. (Sheet 3 of 3)

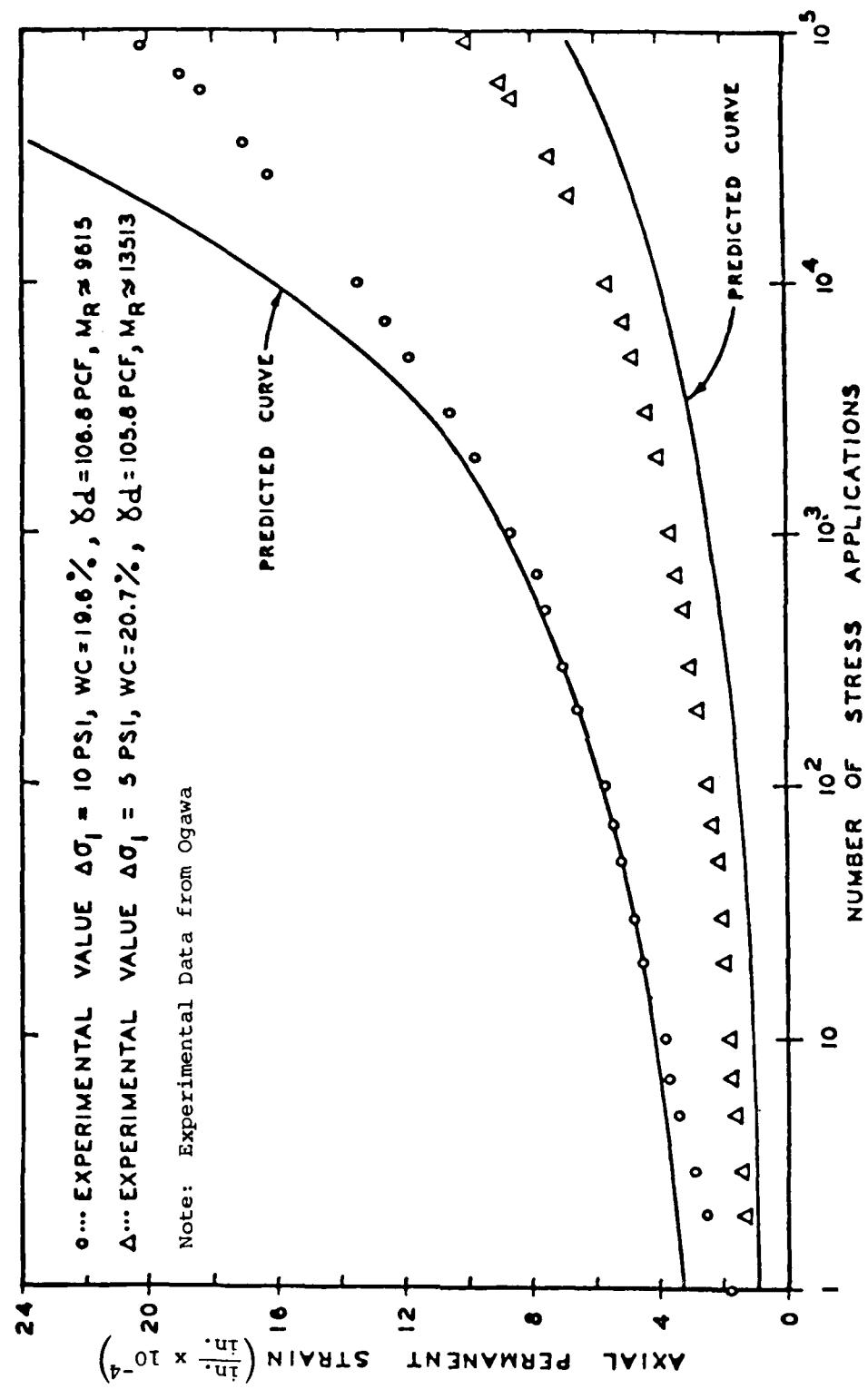


Figure 7. Comparison of permanent strain model with test data

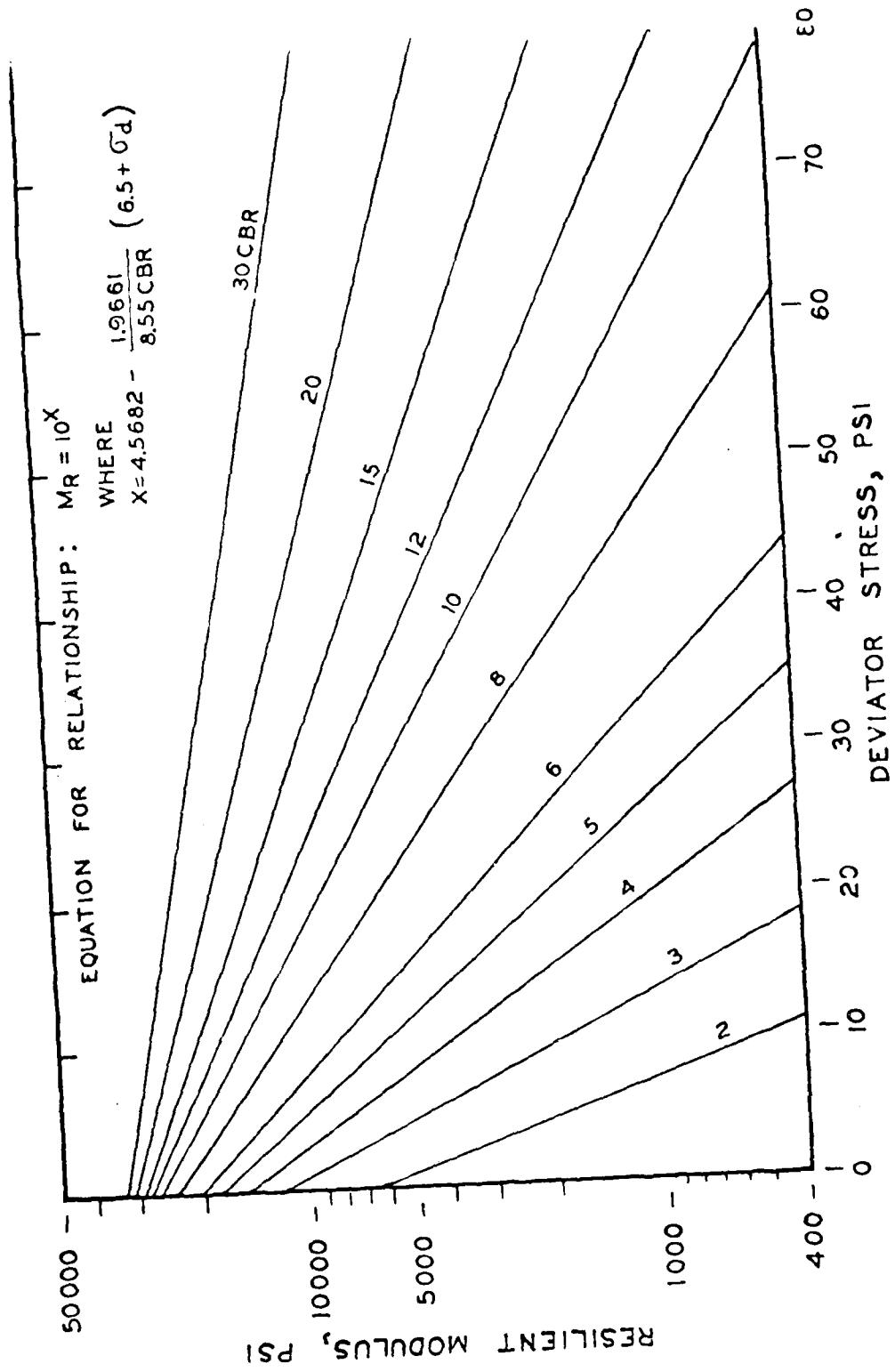


Figure 8. Relationship for predicting M_R from CBR and stress

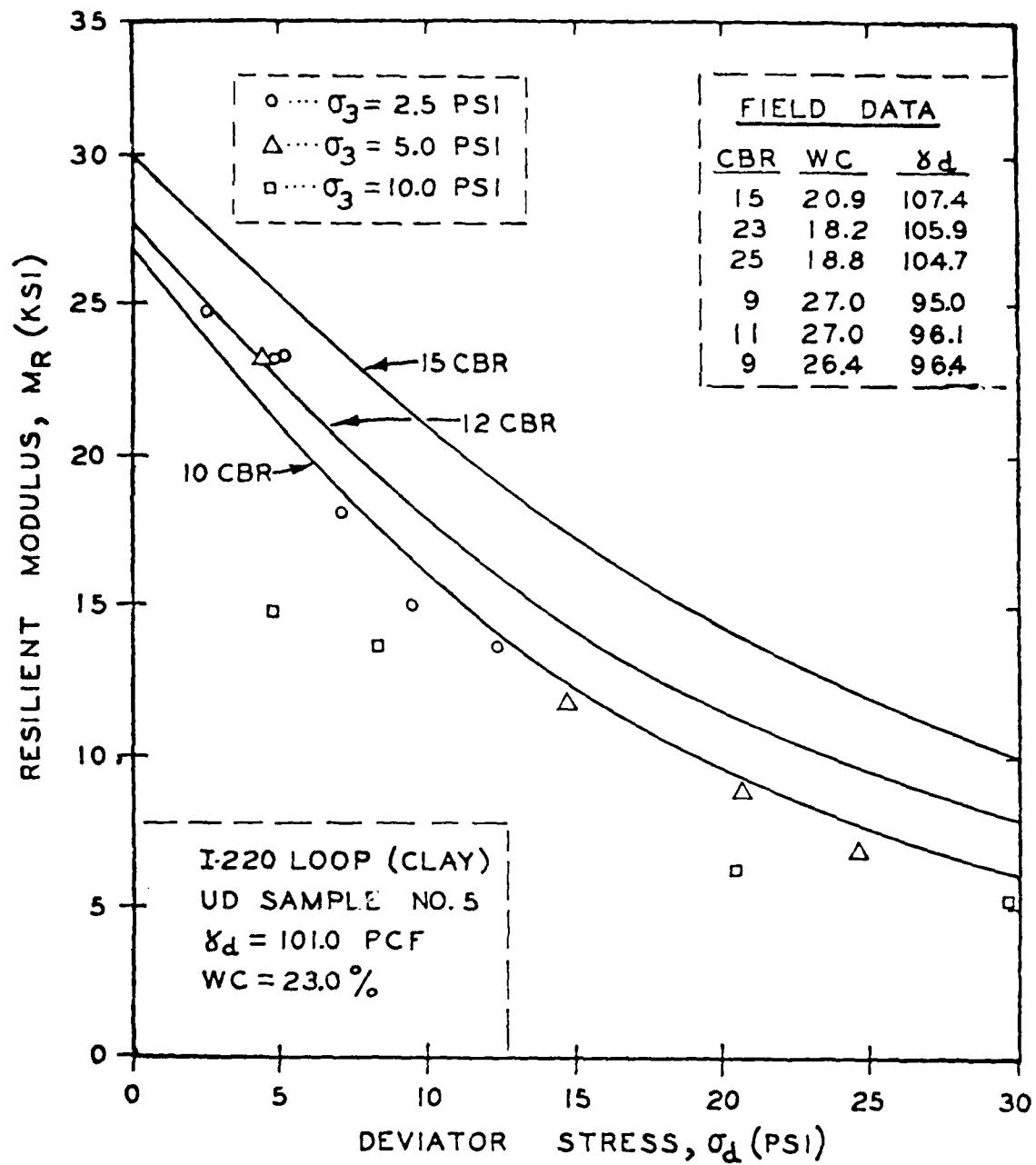


Figure 9. Example resilient modulus data and comparison with model

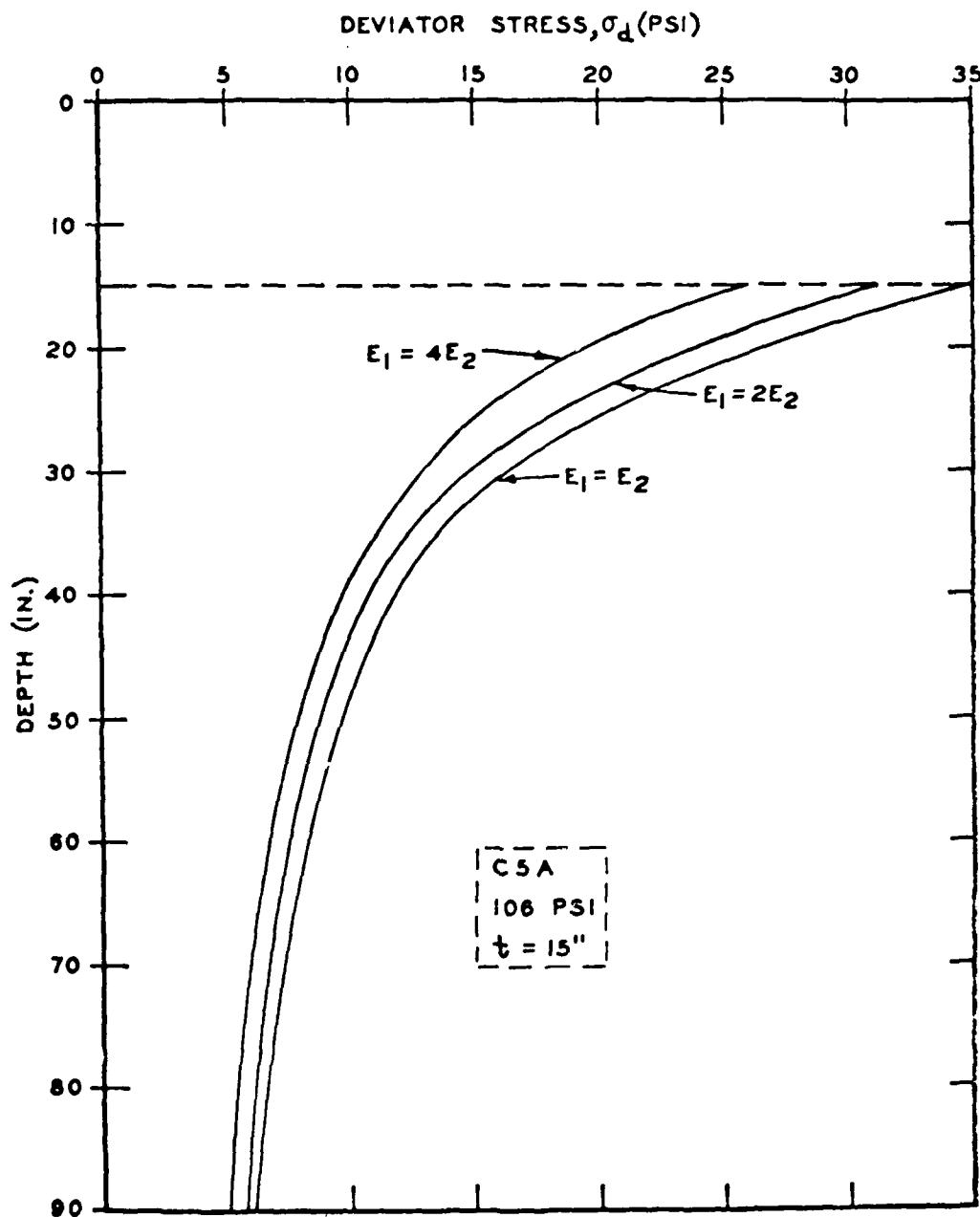


Figure 10. Stress distribution for C-5A for
15-in. structural layer

PROCEDURE FOR COMPUTING PERMANENT DEFORMATION OF SUBGRADE

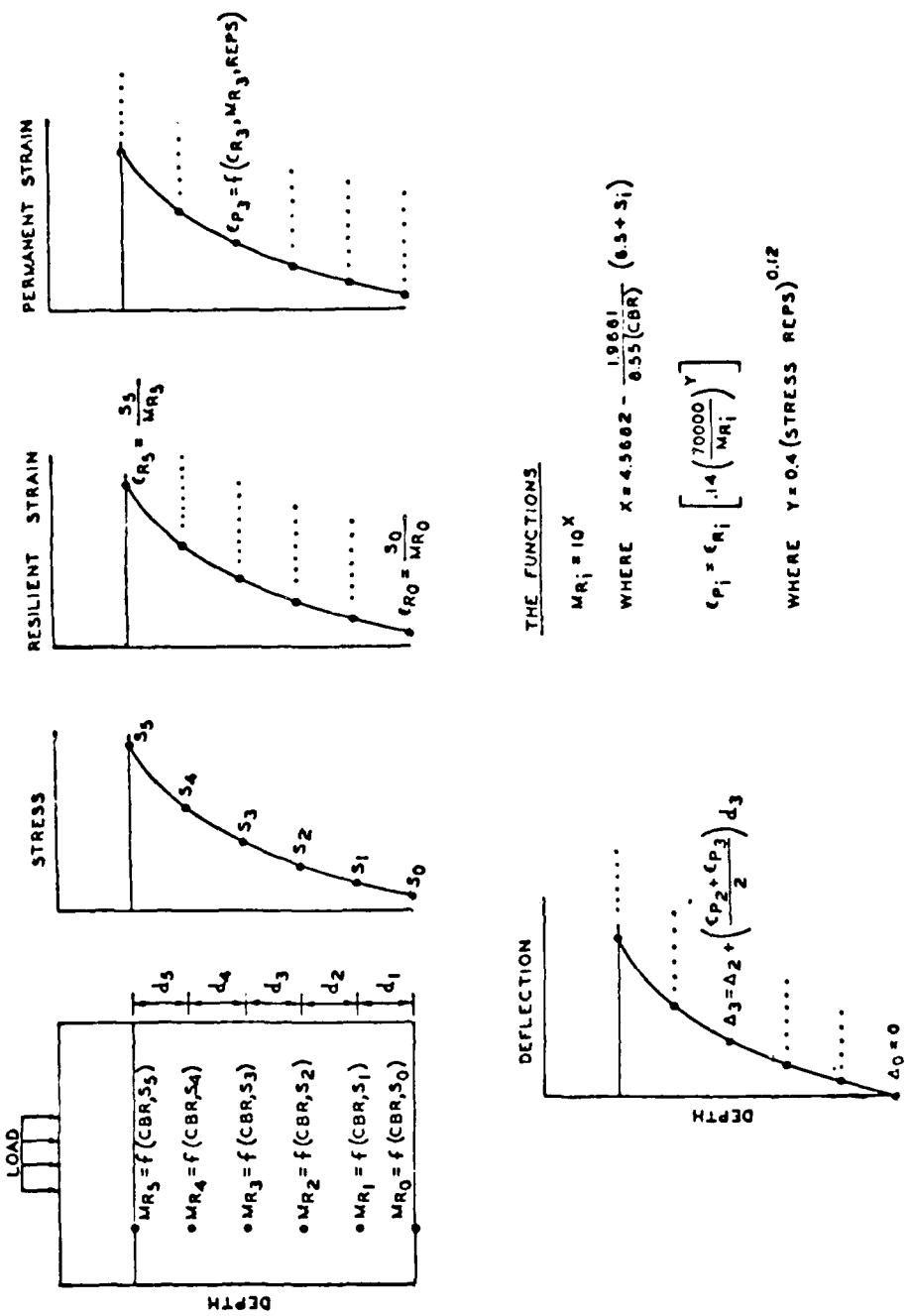


Figure 11. Illustration of rutting model

Appendix A: Program BETADIS

Listing of Program BETADIS

```

100 CHARACTER C*8
101 INPUT ION COUNT, P00
102 INPUT COUNT, ION
1200 INPUT NUMBER OF NUMBER TO GENERATEED
1400 PRINT, "INPUT NUMBER OF NUMBER TO BE GENERATED"
1600 READ, ND
1800 PRINT, "INPUT MEAN, STANDARD DEV., COUNT, COUNT"
1900 READ, CMFAN, CTDEV, COUNT, COUNT
2200 PRINT COUNT, ALPHABETIC TETA
2400 R = CMFAN-CTDEV
2600 MM = COUNT-R***2
2800 R = COUNT-R***2
3000 R = COUNT-R***2-(MM-MM+1+MM)
3200 R = (R+1)*MM - (R+2)
3400 PRINT 901, R, P
360 901 FORMAT(2X, 'ALPHA =', F10, 3, ' BETA =', F10, 3,
380 CALL DTIME(T)
400 D = D + 0.01
420 Y = RAND(D)
440 PRINT, ' ', RAND NUMBER          PARTIAL BETA      BETA NUMBER
460 SUM = 0, 0
480 SUMP = 0, 0
500 TEMP = 0, 0
520 TEMPP = 0, 0
525 DO 50 K = 1, 10
527 COUNT(K) = 0
529 50 CONTINUE
535 ETOTAL = NO / 10
536 T = ETOTAL + 10
537 IF(T,LT,NO), ETOTAL = ETOTAL + 1
540 DO 100 L = 1, NO
545 100 CONTINUE
560 Y = RAND(0, 0)
565 K = Y + 10,
580 K = K + 1
585 IF (COUNT(K), GE, ETOTAL), GO TO 60
590 PRINT, K = PRINT, K + 1
595 TEMP = TEMP + V
600 TEMPP = TEMP + V***2
620 Z = BETA + V*H*B
640 MM = COUNT + V**2
660 PRINT 900, V, MM, MM
680 SUM = SUM + MM
700 SUMP = SUMP + MM *** 2
725 COUNT(K) = MM
730 900 FORMAT(2X, PF15, A)
740 100 CONTINUE
760 AMG1 = TEMP, NO
780      DEM1=COUNT*(TEMPB-1,TEMPB+2),NO,NO,NO
800 AMG = SUM, NO

```

```

820      DEW = .7*TEMP + SUM*(P0 - NO) / (NO)
840      DEW = SORT(ITEM)
860      PRINT 902, AVG, DEW
861      SUM = 0, 0
862      SUMP = 0, 0
863      DO 200 I = 1, NO
864      XXM(I) = XMEAN + (XXM(I) - AVG)*STDEV/DEW
865      IF XXM(I) .LT. XMAX .AND. XXM(I) .GT. XMIN = 1
866      IF XXM(I) .LT. XMIN .AND. XXM(I) = 1
867      SUM = SUM + XXM(I)
868      SUMP = SUMP + XXM(I)**2
869 200 CONTINUE
870      AVG = SUM/NO
871      DEW = (SUMP - (SUM**2)/NO)/NO
872      DEW = SORT(ITEM)
873      PRINT 903, XXM(I), I=1, NO
874      PRINT 902, AVG, DEW
880 902 FORMAT(5X, 'MEAN =', F15.5, ' STANDARD DEVIATION =', F15.5)
881 903 FORMAT(6F12.3)
890 PRINT 902, AVG, DEW
900 STOP
940 END

```

```

1100      RAND1 -- UNIFORM RANDOM NUMBER GENERATOR
1200      OCT 1970
130      FUNCTION RAND(X)
140      TF(X) 1.0-P0,P0
150      20 P0=RHO*RAND1
160      P01=AMOD(P0,BN)
170      RAND=P01/BN
180      RETURN
190      10 P0=7.0**10
200      BN=10.0**10
210      RAND=-X
220      GO TO 20
230      END

```

```

100    BETA
101  *****INCOMPLETE BETA FUNCTION APPROXIMATION*****
200    FUNCTION BETA(M,N,BP=1.0)
300    T=BPM1
400    B=BP*B
500    M=M-1
600    N=N-1
700    T=1.0
800    IF T<0.0 THEN 200
900    C1=BP*B+M-1.0
1000   C2=BP*B+M-0.5
1100   RETURN
1200   RETURN
1300   200  T=(1.0-BP*B)*B
1400   220  T=T*(1.0-BP*B)
1500   230  T=T*(1.0-BP*B)
1600   237  BETA=BETA(M,N-1.0)
1700   240  BETA=BETA(M,N-1.0)
1800   245  B1=BETA(M,N-1.0),BETA
1900   250  C=BTION(M,N,BETA)
2000   255  BETA=BETA+BETA*BTION(M,N,BETA)
2100   260  BTION=M-N-1.0,4.0,4.0
2200   265  T=1.0+BETA*B*B*B
2300   270  B=B
2400   275  B=0.0
2500   280  B0 TO 200
2600   285  BETA=1.0
2700   290  B0 TO 200
2800   295  BTION
2900   300  B0 TO 200
3000   305  BETA=B
3100   310  B0 TO 200
3200   315  BETA=1.0+BP*(1.0-M)
3300   320  B0 TO 200
3400   325  BETA=BP*(1.0-M)
3500   330  B0 TO 200
3600   335  BTION
3700   340  BTION(BETA,M,N,B)
3800   345  BTION(M,N,B)=0.0
3900   350  DATA M1, 04250425, 14020179, 22900829, 369493812,
4000   355  , 32270300, 43281540, 472829791, 49470647
4100   360  DATA N1, 09472571, 09120171, 09457521, 07479799,
4200   365  , 08231449, 04797921, 02112678, 01757422
4300   370  TIME=0.0
4400   375  B0 TO 200
4500   380  B1=B1+C1*(T+B)*(T+B)
4600   385  B1=B1+C1*(T+B)*(T+B)
4700   390  C1=BTION(M,N,B1)
4800   395  BTION=M-N-1.0,B1
5000   400  BTION=M-N,B1
5100   405  BETA=B1+C1*TIME
5200   410  BTION
5300   415  END

```


List of Numbers Having Beta Distribution
Obtained from Program BETADIS

6.990	7.970	12.081	6.575	4.092	4.002
11.527	5.810	10.030	8.418	6.230	6.926
7.261	3.965	10.229	9.701	6.806	6.129
5.911	3.787	7.597	4.393	5.418	4.947
5.145	8.675	8.030	3.653	4.360	3.889
7.016	9.378	5.388	4.659	7.681	7.325
5.264	5.327	4.810	8.705		

Mean = 6.7; Standard Deviation = 2.2

Appendix B: Development of a Basic Rutting Model

INTRODUCTION

The Waterways Experiment Station (WES), in seeking a more rational approach to design of flexible pavements than the presently used CBR design procedure, has developed, under sponsorship of the Federal Aviation Administration (FAA) and the Office, Chief of Engineers (OCE), subgrade strain criteria for both roads (Brabston, et al.¹) and airports (Barker and Brabston²). The criteria now being incorporated into both FAA and OCE design procedures are considered to be the basis for limiting the rutting of flexible pavement. The approach of limiting the resilient strain in the subgrade in order to limit rutting of the subgrade implies that a relationship exists between the permanent strain and the resilient strain of subgrade soils. In both of the design procedures the limiting subgrade strain criteria were presented as a function of the subgrade modulus. The author² discussed this point in detail and presented the results of laboratory tests which indicated that the relationship between the permanent strain and resilient strain was indeed a function of the stiffness of the material. The work in developing these strain criteria and in studying the results of laboratory tests for conformation of the criteria created an interest in the relationship between the permanent strain and resilient strain of subgrade soils.

SUBGRADE STRAIN CRITERIA

A comparison of the different subgrade criteria is presented in Figure 1. All of the criteria presented with the exception of that developed by Dr. Chou (Reference 3) has been developed from pavement sections conforming to some

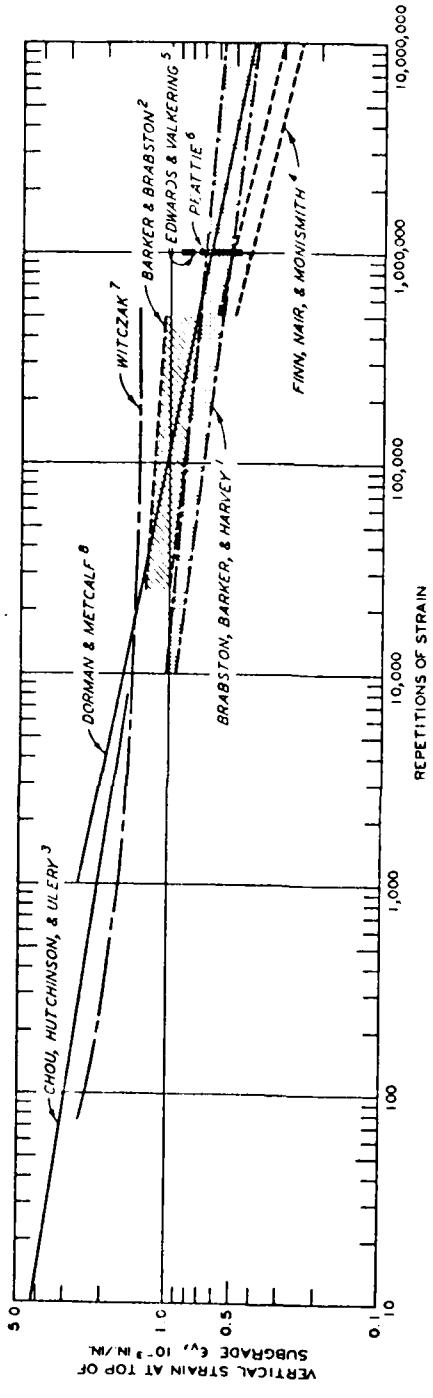


Figure B1. Comparisons of subgrade strain criteria

previously established design standards. Although different design standards may have been used and different methods were employed in characterizing the granular materials, it is seen when grouped that all of the criteria form a relatively narrow band across a wide range of repetition levels. Consider that for repetition level of 1×10^5 repetitions the range of the criteria is from 0.15×10^{-3} to 1.4×10^{-3} in./in. Even if the criteria of Finn, et al.,⁴ and Chu⁵ were extrapolated to this level of repetition, the range would not be increased. From the evidence it would appear that for a given level of repetitions the resilient strain at which the permanent strain becomes unacceptable would be within a fairly narrow band. The data presented in Figure 1 strongly indicate a unique relationship between permanent strain and resilient strain. Considering the emphasis being placed on the use of the repeated load triaxial test in which both the resilient and permanent strains are measured, it would seem that the concept of limiting subgrade strain criteria could be substantiated or disputed from the results of such laboratory tests.

LABORATORY TEST

In recent years much interest has been generated principally by the work of Barksdale⁹ in the use of the repeated load triaxial test as a method to predict rutting of a pavement. The aim of nearly all of the laboratory rutting tests performed to date has been: first, to define the permanent deformation as a function of the applied stress; and second, to define the resilient modulus of the material as a function of stress. Only one experiment known to the author has been performed with the first objective of defining the relationship between resilient strain and permanent strain. This experiment reported by Chisolm¹⁰ was conducted to substantiate the concept by the author that the allowable resilient subgrade strain is a function of modulus of the

subgrade; i.e., to establish the relationship between resilient strain and permanent strain for soils having different moduli. To accomplish this objective, a Vicksburg heavy (buckshot) clay (CH ani E-11) was molded at four different water contents and tested in a repeated load triaxial test. Since the object of the experiment was to determine only the relative relationships between resilient strain and permanent strain between the samples, only 1000 load repetitions were applied to each sample. The results of the experiment are shown in Figure 2. Another experiment in which the establishment of the relationships between resilient and permanent strain was only a minor part was conducted at the University of California at Berkeley by Ogawa (Reference 11). The results of this experiment in regard to the relationship between resilient strain and permanent strain are shown in Figure 3. The results of these tests are close to those of WES, even though the soils are different and the tests were conducted at different strain levels and a different number of strain repetitions were applied. Both Chisolm and Ogawa used almost identical equipment and procedures in conducting the two experiments. One seemingly important factor is that both used LVDT holding clamps placed on the specimen (a description of the measuring device is given in References 10 and 11) for measuring both resilient and permanent strains. When examining the results of other researchers, the evidence indicates a distinct difference in the test results depending on how the strains are measured.

The majority of research reports studied were written primarily to present the permanent or the resilient deformation characteristics of a material but not the relationship between the two. Thus, in order to develop these data, in many cases it was necessary to calculate missing parameters which, in most cases, was the resilient strain. The data studied were that reported by Barksdale,⁹ Kalcheff and Hicks,¹² Fossberg,¹³ Feed and McNeill,¹⁴ Brown et al.,¹⁵

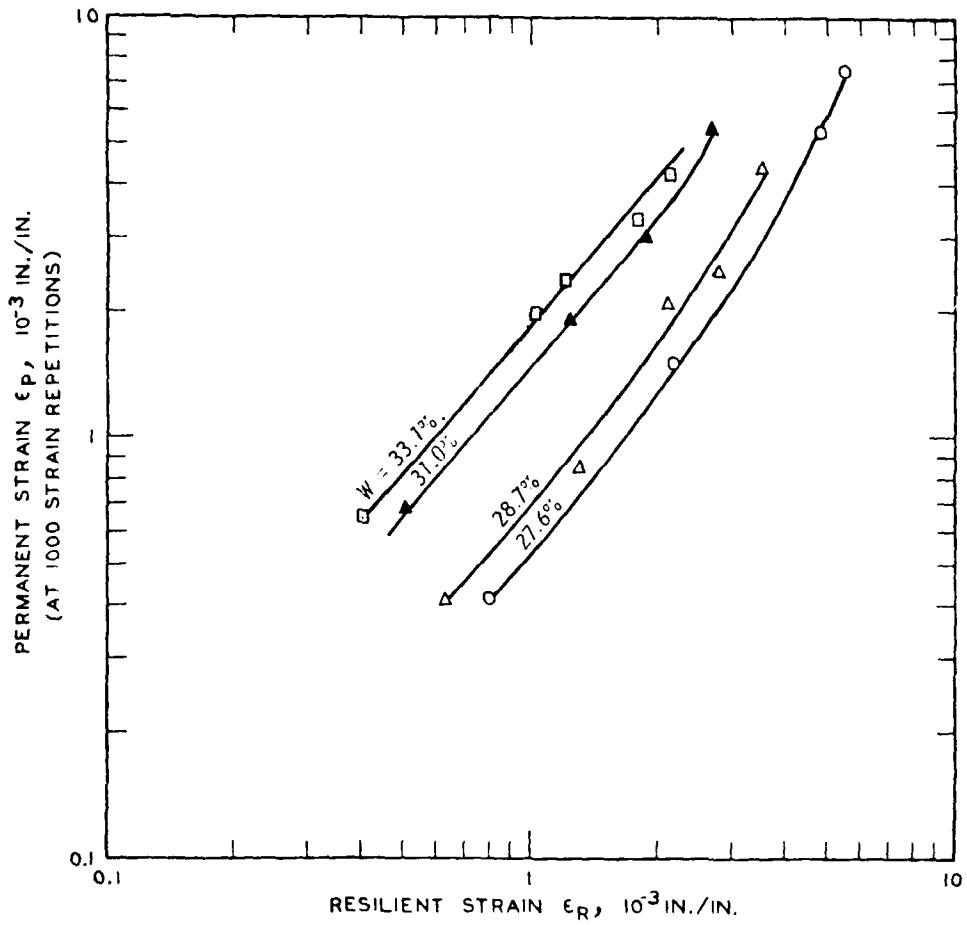


Figure B2. Relationships between permanent strain and resilient strain for buckshot clay

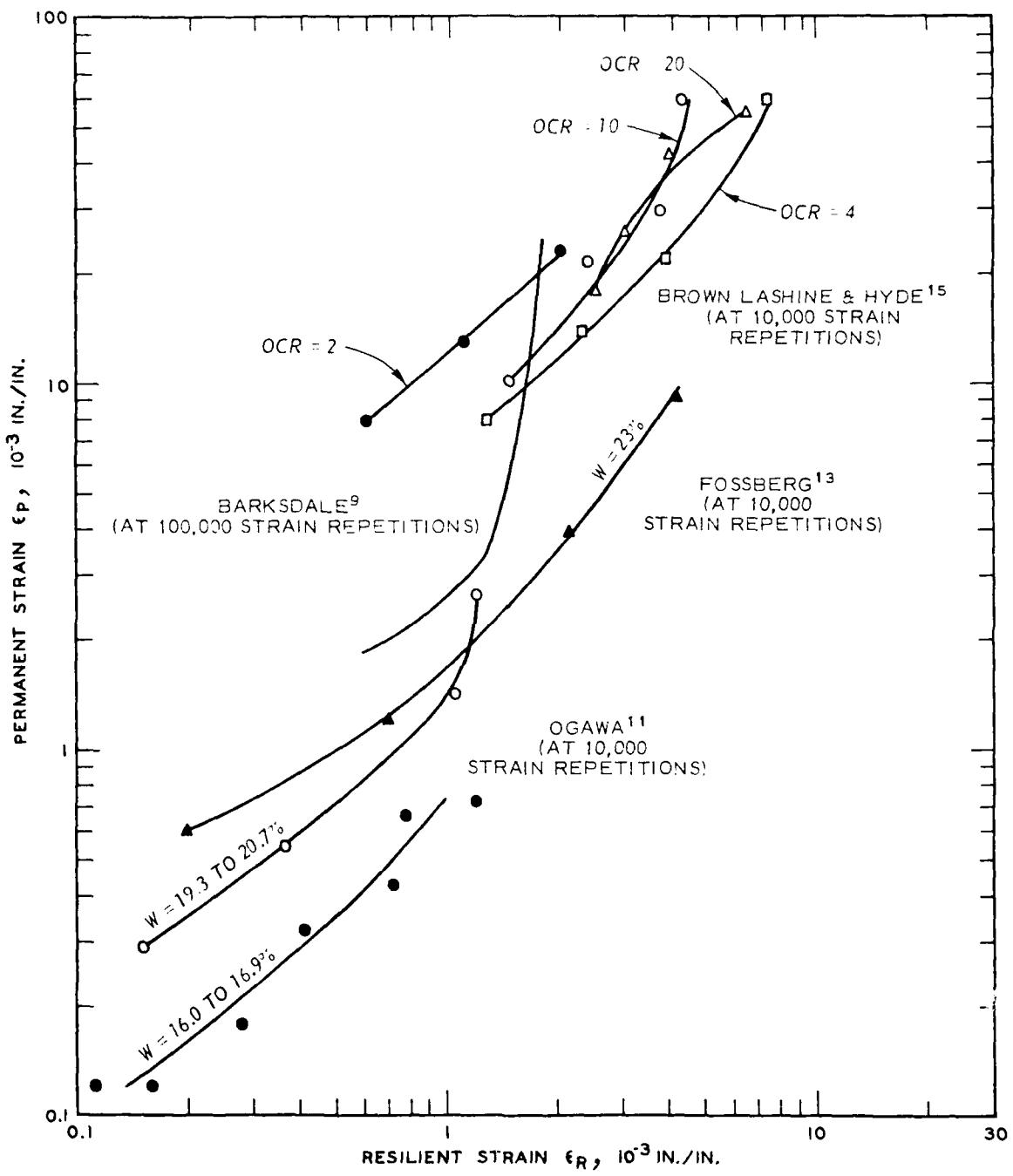


Figure B3. Relationship between permanent strain and resilient strain for different materials

Kirwan et al.,¹⁷ and Chiscolm.¹⁸

The experiment involving the most types of soil for which the relationships could be established was that conducted by Kirwan. The soils used in the experiment are listed in Table 1. This experiment was not conducted in a manner which would yield data compatible with the objective of the Chiscolm experiment. Basically the tests were conducted at two stress levels for each soil strength. In each soil, different water contents were employed in order to obtain a wide range of permanent deformations. This procedure is in contrast to Chiscolm's procedure in which the stress was varied to obtain a range of resilient strains. Thus, for a given soil strength, only two data points were available to obtain a relationship between the resilient and permanent strains for a given soil strength; therefore, the data were combined as shown in Figure 4. When presented in this manner, it appears that the data forms no relationship for all the soils except soil No. 3. It should be remembered that the test points for the higher strains were obtained by increasing the water content and would therefore represent data from samples of lower strength. If the soils in this experiment behave similarly to those tested by Chiscolm and Ogawa, the permanent strain for a particular resilient strain would be higher for the soil at the higher water content but lower stress than for the soil at the lower water content but higher stress. The effect of the varying of the water content to obtain higher resilient strains would be to produce a steeper relationship than would have been obtained by maintaining a constant water content and increasing the applied stress to obtain higher resilient strains.

Another interesting but difficult set of data to interpret was the data presented in Reference 19. In this experiment, the effect of stress history of the samples on permanent deformation and resilient strain were

TABLE B1
(From Reference 1C)

Soils Used in the Investigation

<u>Clay No.</u>	<u>Soil Type</u>	<u>Location</u>	<u>Liquid Limit</u>	<u>Plastic Limit</u>	<u>Plasticity Index</u>
1	CL	Lucan	29 percent	22 percent	7
2	CL	Wexford	30 percent	16 percent	14
3	CH	Coal Island	67 percent	25 percent	42
4	CH	Dublin	35 percent	19 percent	16
5	CH	Devon (TA)	63 percent	23 percent	40
6	CH	Darndale (A)	52 percent	27 percent	25
7	CL	Darndale (E)	46 percent	27 percent	19
8	CL	Darndale (F)	39 percent	22 percent	17
9	CL	Darndale (G)	33 percent	18 percent	15

Soil type refers to the Unified Soil Classification System.

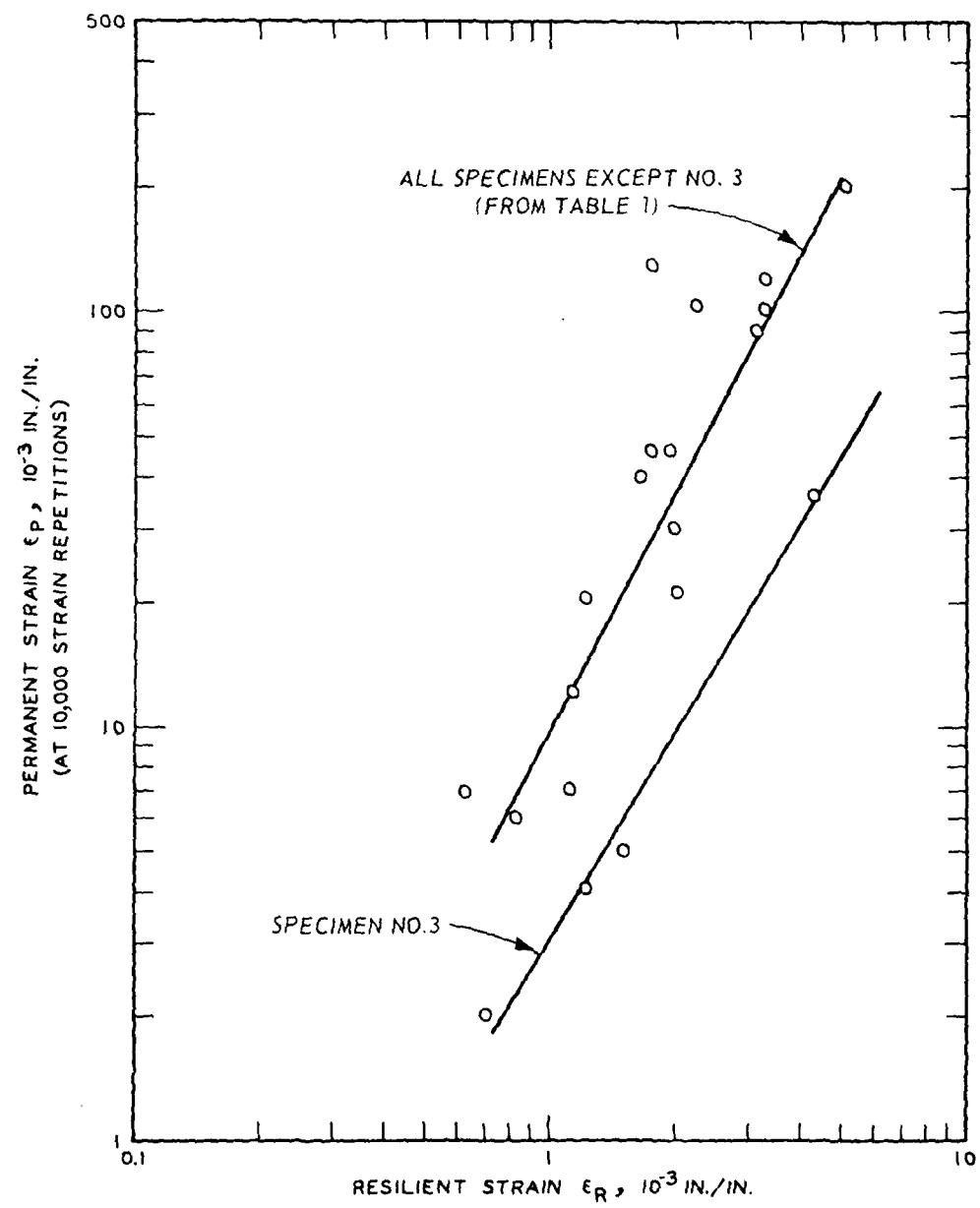


Figure B4. Relationships between permanent strain and resilient strain from data developed by Kirwan et al.¹⁶

stirified. Although it was concluded in the paper that stress history had an effect on the permanent strain but no effect on resilient strain, the data as plotted in Figure 3 indicated the relationship between permanent strain and resilient strain is little affected by moderate differences in stress history. In the plot, a distinct relationship for each overconsolidation ratio is presented but the difference appears inconsistent and could be experimental error. A single relationship would probably suffice for this set of data. The data presented by Barksdale (Figure 3), Sved (Figure 5), and Fossberg (Figure 3) are all straightforward, each of which provided useful information. Of all the tests, those conducted by Fossberg covered the largest range (.0002 to .0042 in./in.) of resilient strains. Two tests, one by Barksdale and one by Ogawa, indicated very abrupt change in the slope of the relationship. For the test by Barksdale the abrupt change came at a resilient strain of .0013 in./in. and for Ogawa at .0011 in./in. A more gradual change at about the same magnitude of resilient strain was indicated in the results presented by Fossberg. Such behavior lends strong support to the use of the limiting subgrade strain concept in pavement design. It can also be pointed out that the strain criteria developed to date (presented in Figure 1) are consistent with the laboratory results.

In examining the data plots, it was noted that results of Chisolm, Fossberg, and Ogawa group together and the data of the other researchers fall into another group. It has already been emphasized that Chisolm and Ogawa employed inside LVDT holding clamps attached to the specimen to measure both resilient and permanent strains. Fossberg also used the same devices for the tests he conducted. Barksdale found that such measuring devices gave him inconsistent readings of permanent deformation and therefore he employed outside measuring devices for measuring permanent strain but still used LVDT

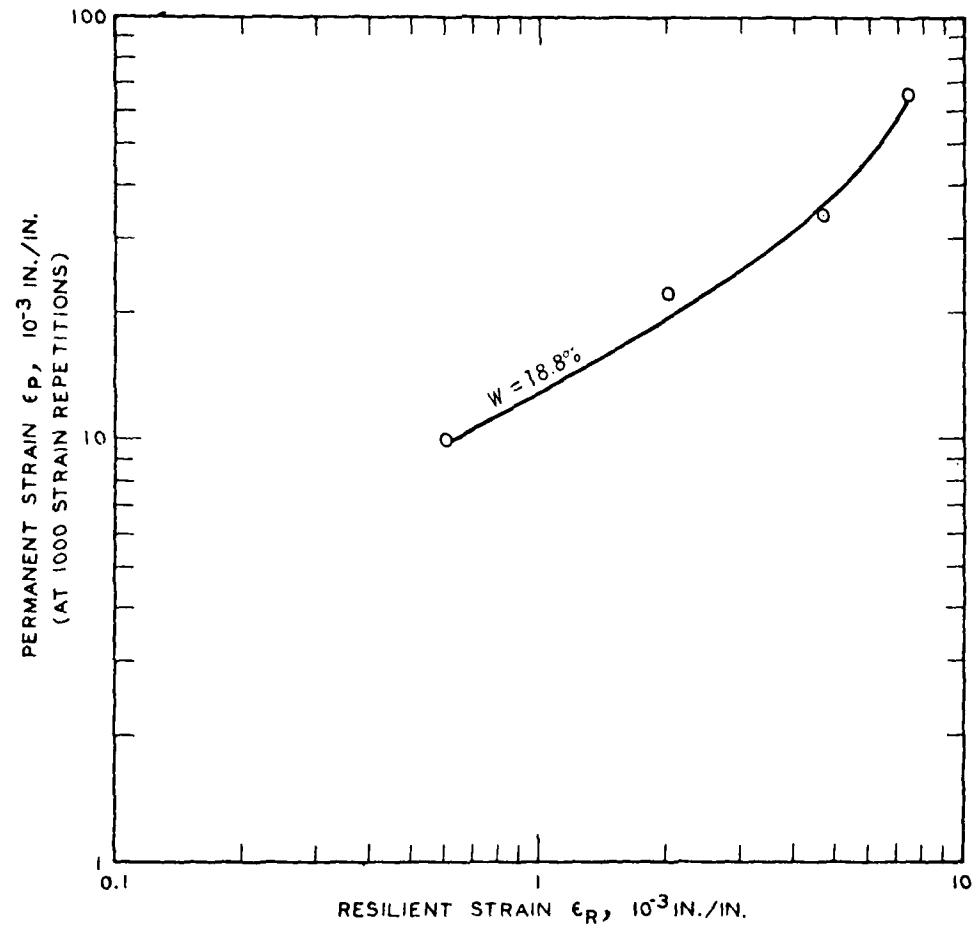


Figure B5. Relationship between permanent strain and resilient from data presented in Reference 14

clamps for measuring resilient strains. It is believed that the other researchers all used outside measuring devices which measured the deflection over the entire sample. Thus, the data fall into two groups; that in which the permanent strain was measured using LVDT clamps and thus only over the center portion of the sample, and that in which the permanent strain was measured using outside devices and thus measured the strain over the entire sample. The difference in the two groups of data is almost an order of magnitude.

Additional data are available by including tests on granular materials. Such data have been presented by Chisolm, Barksdale, and Kalcheff and Hicks. The plot of these data is shown in Figure 6. These data indicate a steeper relationship between resilient strain and permanent strain than was indicated for subgrade soils. Considering the great difference between the material properties, the relationships for the granular material are surprisingly close to those for subgrade soils. It is quite possible that the limiting strain concept could be extended to apply to granular subbase and base materials.

USE OF LABORATORY DATA

It has been shown that the repeated load triaxial test is a method for developing the relationship between resilient strain and permanent strain. There is still some question as to the best procedure for measuring the permanent strain and that the results obtained will depend on the particular procedure used.

In addition to establishing or verifying limiting strain criteria the data presented in this manner can also be used to estimate the permanent deformation of the subgrade. In a report on the structural analysis

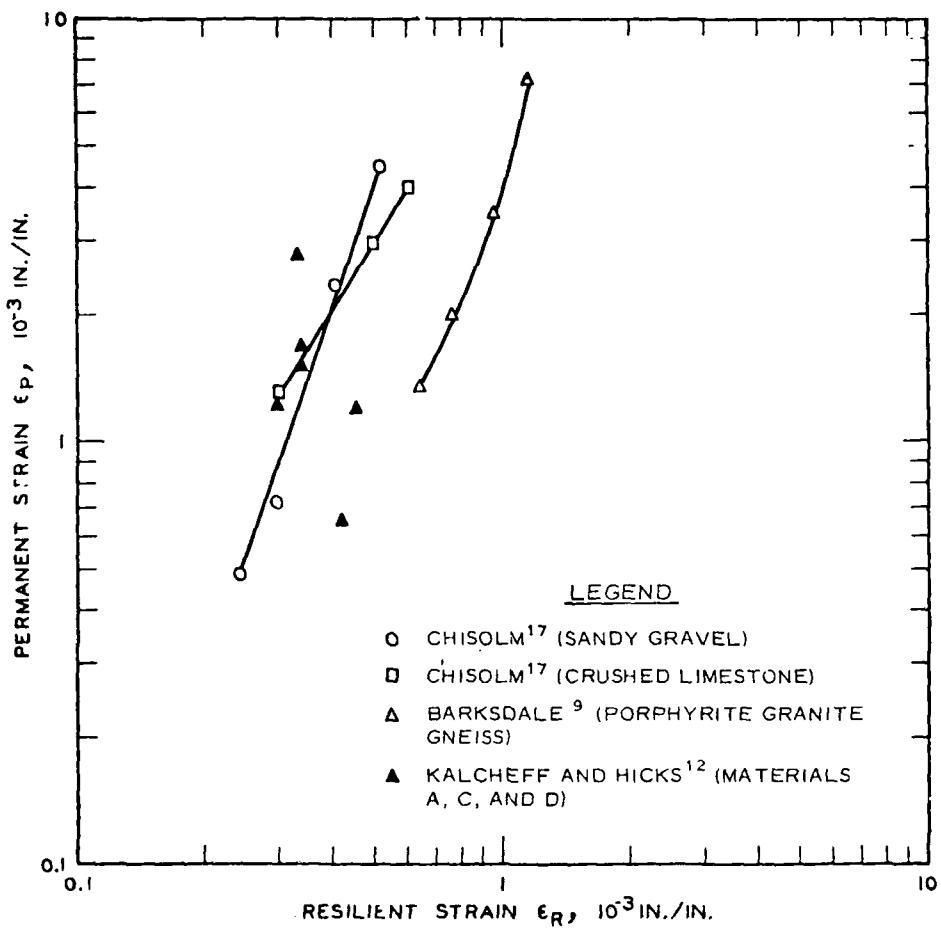


Figure B6. Relationships between permanent strain and resilient strain for granular materials

(Reference 18) of insulated layers the author used the WEC test data to show that the rutting of the subgrade was an insignificant part of the total rutting of the pavement. In the analysis the distribution of the computed vertical resilient strain within the subgrade is shown in Figure 7. Using this distribution of resilient strain and the relationship between resilient strain and permanent strain, as established by Chiselm, then the distribution of permanent strain as shown in Figure 7 was determined (that is, $\epsilon_p = \epsilon_R \left(\frac{\epsilon'_p}{\epsilon'_R} \right)$), where

ϵ_p = the computed permanent strain

ϵ_R = the computed resilient strain

ϵ'_p = measured permanent strain in repeated load triaxial test

ϵ'_R = measured resilient strain in repeated load triaxial test

The total permanent deformation (Δ_p) was then determined by assuming the permanent strain went to zero at a depth of 120 in. below the top of the sub-

grade and computing the area under the curve; i.e., $\Delta_p = \int_{120}^0 \epsilon_R \left(\frac{\epsilon'_p}{\epsilon'_R} \right)$. Using

this procedure, the deformation at the top of the subgrade was estimated to be 0.08 in.

A conservative estimate of the permanent deformation at the top of the subgrade can be estimated quickly and easily by using the ratio of permanent strain to resilient strain as determined from the computed resilient strain at the top of the subgrade. The ratio is then used as a constant multiplier to the computed subgrade resilient deformation (Δ_R) to estimate the subgrade permanent deformation. This assumes that the ratio of permanent strain to resilient strain remains constant with depth which, of course, is not true.

This is to say if $\frac{\epsilon'_p}{\epsilon'_R}$ is a constant, then the previous equation becomes

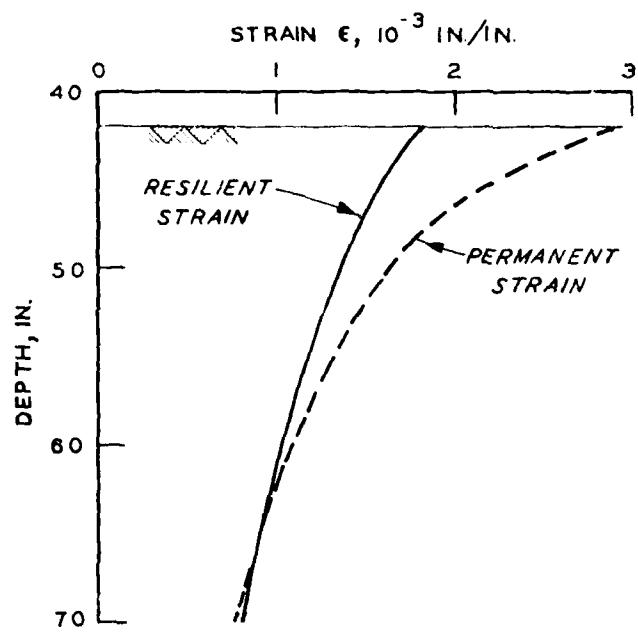


Figure B7. Computed distribution of resilient and permanent strain in the subgrade of a flexible pavement test section

$\Delta_p = \frac{\epsilon'_p}{\epsilon'_R}$, $\int_0^{\infty} \epsilon'_R = \frac{\epsilon'_p}{\epsilon'_R} \Delta_R$. For the example given above the strain ratio is about 1.6 and the computed resilient deformation at the top of the subgrade was 0.115, giving an estimated permanent deformation of 0.184. This estimate is over twice the previous estimate. The difference being, as mentioned before, that the maximum strain was assumed constant with depth and also that previously it was assumed that the permanent deformation was zero at 120 in., whereas in the latter estimate it was assumed the permanent strain extends to an infinite depth.

The same procedure can be used to compute the permanent deformation in other layers of the pavement systems. Consider the three pavement types as shown in Figure 8 in which sections 1 and 2 were subjected to simulated traffic of a C-5A aircraft and section 3 to the simulated traffic of a 747 aircraft. To compute resilient strains the material properties as shown in Figure 8 were assumed (from procedure given in Reference 2) and a modified version of the Chevron computer program was employed as a mathematical model for computing resilient strains. From the previously discussed laboratory data the relationship of permanent to resilient strain for levels of 100, 1000, and 10,000 strain repetitions as shown in Figure 9 was assumed. These relationships must be considered as a pure guess at the true relationships between the computed resilient strains and the resulting permanent strains in the pavement system. The computations for determining the permanent strains are shown in Tables 2, 3, and 4, and resulting distributions of permanent strain with depth for repetition levels of 100, 1000, and 10,000 coverages are shown in Figures 10, 11, and 12 for pavement sections 1, 2, and 3, respectively. It is to be noted that it has been assumed that no permanent strain occurred in the asphalt concrete and that one coverage produces one strain repetition. It

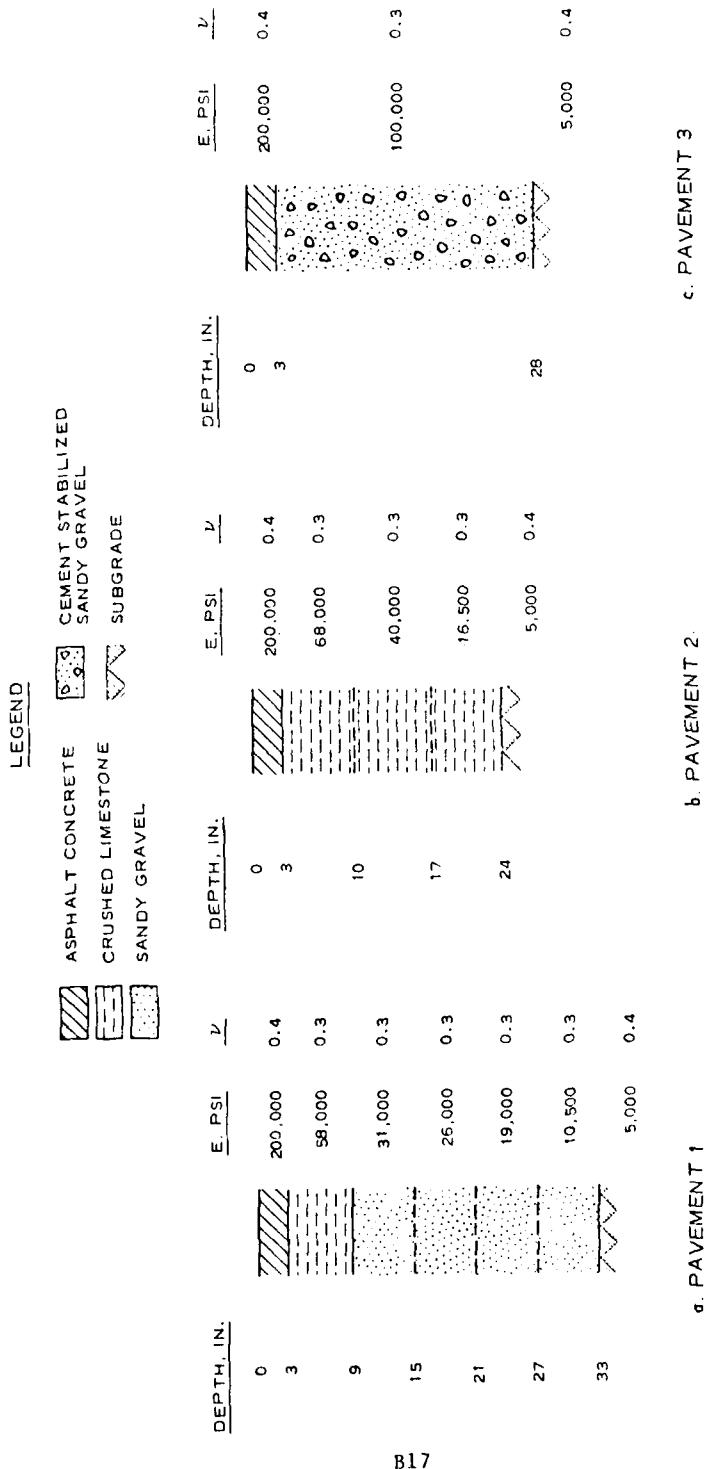


Figure B8. Pavement sections and material properties
(References 19, 20, and 21, respectively)

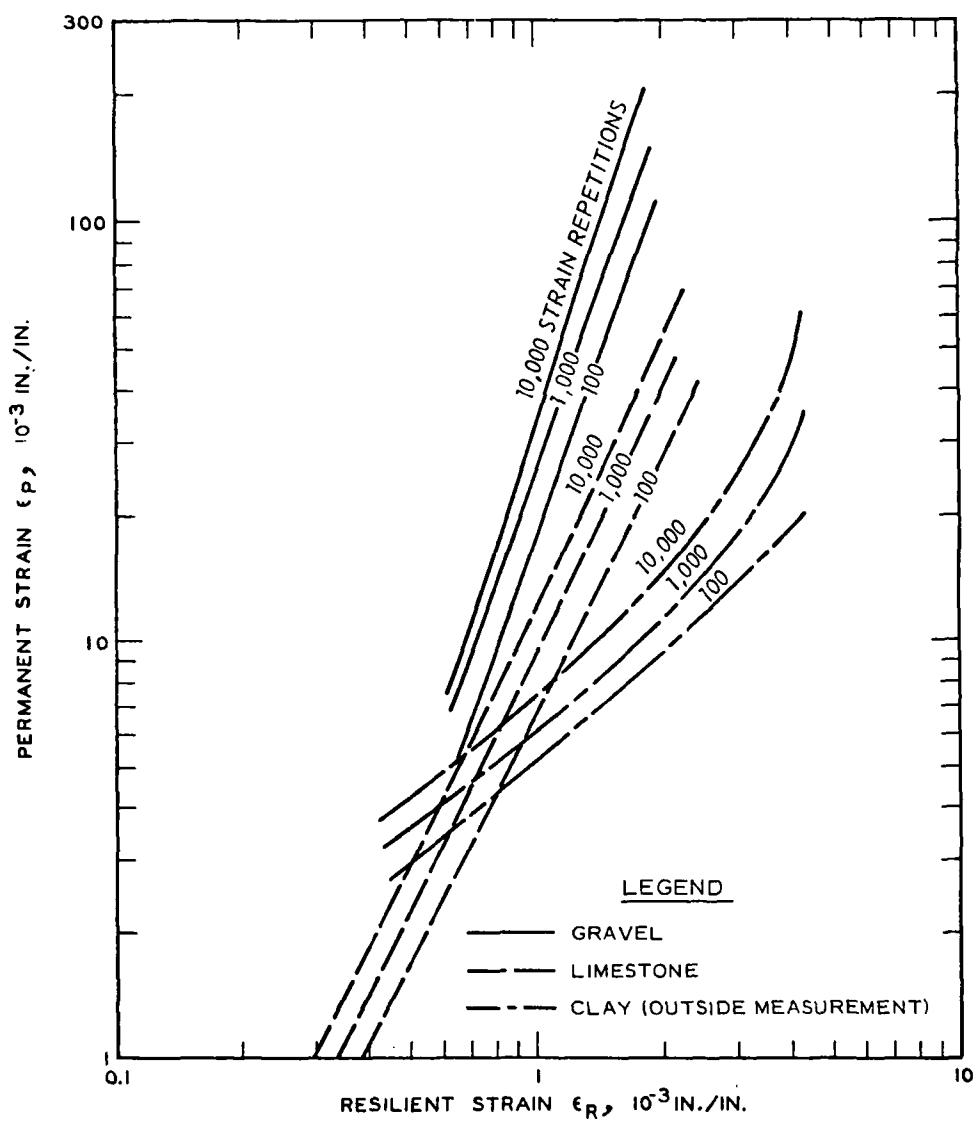


Figure B9. Assumed relationship between permanent strain and resilient

TABLE B2
Computations of Permanent Strain for Pavement 1

<u>Material</u>	<u>Layer</u>	<u>Depth</u>	<u>ϵ_R</u>	<u>ϵ_R</u>	<u>ϵ_p @ 100</u>	<u>ϵ_p @ 1000</u>	<u>ϵ_p 10,000</u>
Cr. St.	2	3	.219	.0015	.0115	.0165	.022
Cr. St.	2	-9	.212	.0011	.0082	.012	.0155
Sand Gr.	3	0	.212	.0016	.068	.10	.145
Sand Gr.	3	12	.208	.0013	.038	.054	.076
Sand Gr.	3	-15	.204	.0010	.018	.026	.034
Sand Gr.	4	15	.201	.0012	.030	.044	.060
Sand Gr.	4	18	.201	.0010	.018	.026	.034
Sand Gr.	4	-21	.198	.0009	.014	.020	.025
Sand Gr.	5	21	.198	.0010	.018	.026	.034
Sand Gr.	5	24	.196	.0009	.014	.020	.025
Sand Gr.	5	-27	.192	.0009	.014	.020	.025
Sand Gr.	6	27	.192	.0012	.030	.044	.060
Sand Gr.	6	30	.189	.0011	.024	.035	.045
Sand Gr.	6	-33	.186	.0011	.024	.035	.045
Clay	7	33	.186	.0016	.0776	.0091	.0115
Clay	7	36	.181	.0016	.0074	.0086	.0109
Clay	7	48	.166	.0012	.0060	.0071	.0085
Clay	7	60	.153	.0010	.0052	.0061	.0074
Clay	7	120	.108	.0006	.0034	.0041	.0048

TABLE B3
Computation of Permanent Strain for Pavement 2

<u>Material</u>	<u>Layer</u>	<u>Depth</u>	<u>Δ_p</u>	<u>ϵ_R</u>	<u>ϵ_p @ 100</u>	<u>ϵ_p @ 1000</u>	<u>ϵ_p @ 10,000</u>
Cr. St.	2	3	.228	.0011	.0082	.012	.0155
Cr. St.	2	6.5	.225	.0010	.0068	.0097	.0126
Cr. St.	2	-10	.222	.0009	.0056	.0078	.0100
Cr. St.	3	10	.222	.0012	.0098	.014	.019
Cr. St.	3	13.5	.218	.0009	.0056	.0078	.0100
Cr. St.	3	-17	.215	.0009	.0056	.0078	.0100
Cr. St.	4	17	.215	.0013	.0115	.0165	.022
Cr. St.	4	20.5	.211	.0011	.0082	.012	.0155
Cr. St.	4	-24	.207	.0011	.0082	.012	.0155
Clay	5	24	.207	.0020	.0094	.0113	.0148
Clay	5	30	.196	.0017	.0082	.0096	.0123
Clay	5	36	.186	.0015	.0074	.0086	.0109
Clay	5	48	.170	.0012	.0060	.0071	.0088
Clay	5	60	.158	.0010	.0052	.0061	.0074
Clay	5	120	.108	.0006	.0034	.0041	.0048

TABLE B4
Computation of Permanent Strain for Pavement 3

<u>Material</u>	<u>Layer</u>	<u>Depth</u>	<u>Δ_R</u>	<u>ϵ_p</u>	<u>ϵ_p @ 100</u>	<u>ϵ_p @ 1000</u>	<u>ϵ_p @ 10,000</u>
Stab. Gr.	1	3	.226	.0013	.0015	.0165	.022
Stab. Gr.	2	5	.220	.0011	.0062	.012	.0155
Stab. Gr.	2	13	.216	.0007	.0034	.0046	.0060
Stab. Gr.	2	18	.213	.0005	.0017	.0023	.0030
Stab. Gr.	2	23	.211	.0004	.0011	.0014	.0019
Stab. Gr.	2	28	.209	.0003	.0017	.0023	.0030
Clay	3	28	.209	.0016	.0076	.0391	.0115
Clay	3	30	.205	.0015	.0074	.0356	.0109
Clay	3	36	.197	.0014	.0069	.0351	.0100
Clay	3	46	.181	.0012	.0060	.0371	.0089
Clay	3	60	.168	.0010	.0052	.0361	.0074
Clay	3	120	.108	.0006	.0034	.0041	.0048

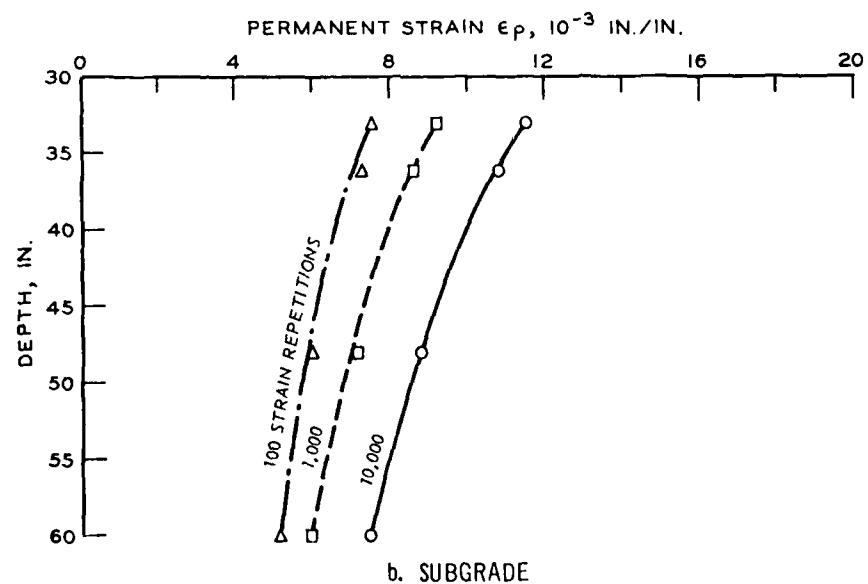
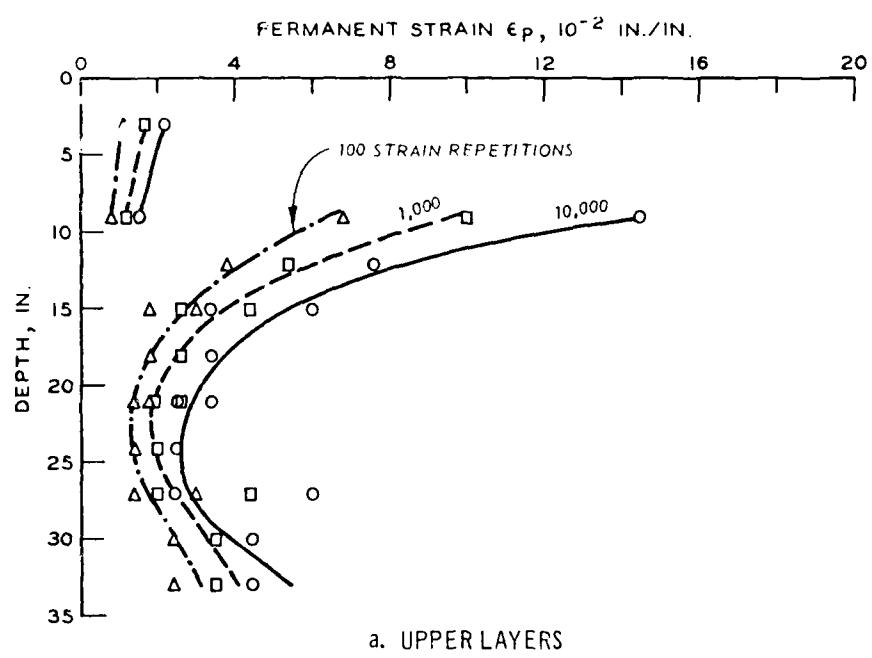


Figure B10. Computed permanent strain for pavement section 1

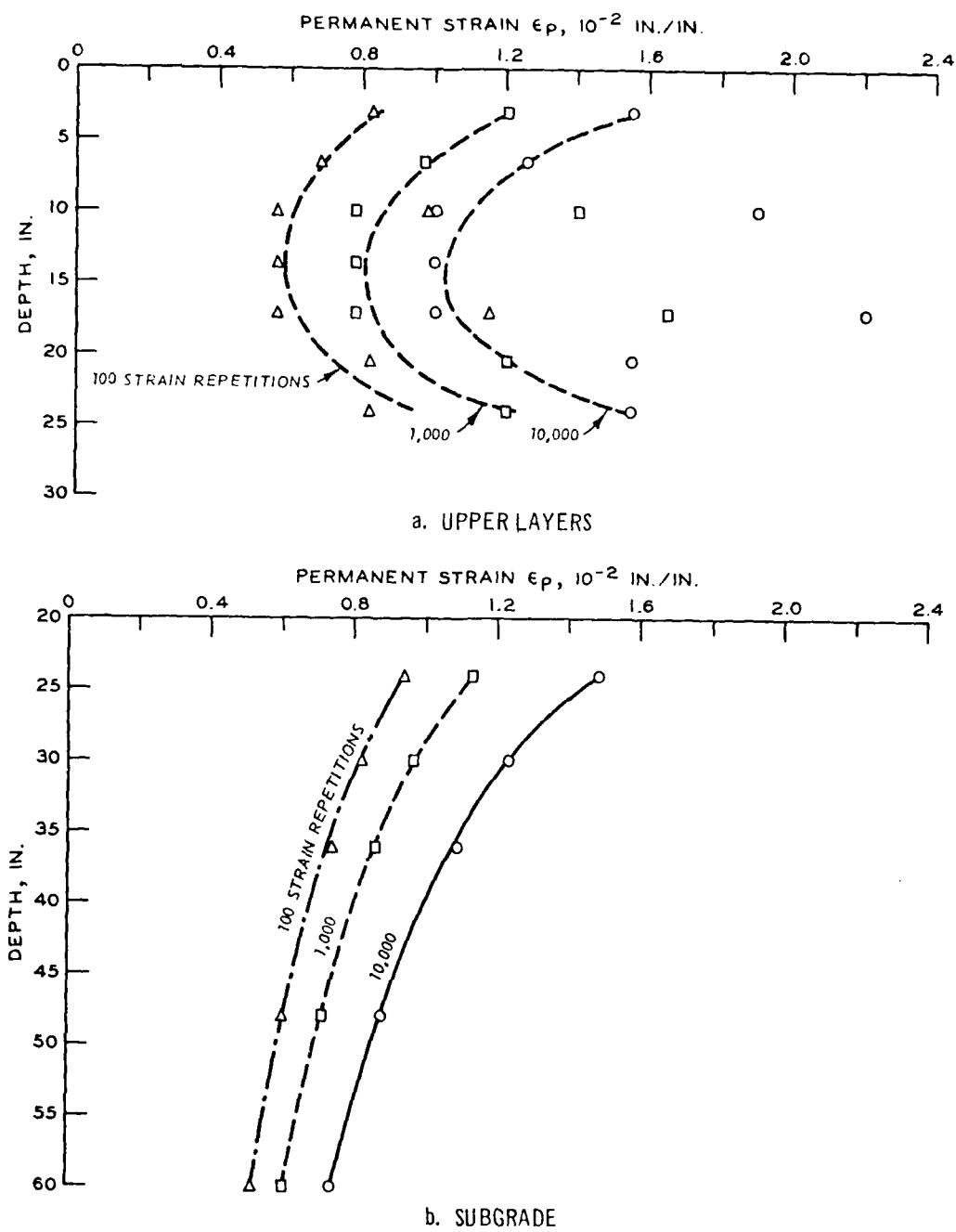


Figure B11. Computed permanent strains for pavement section 2

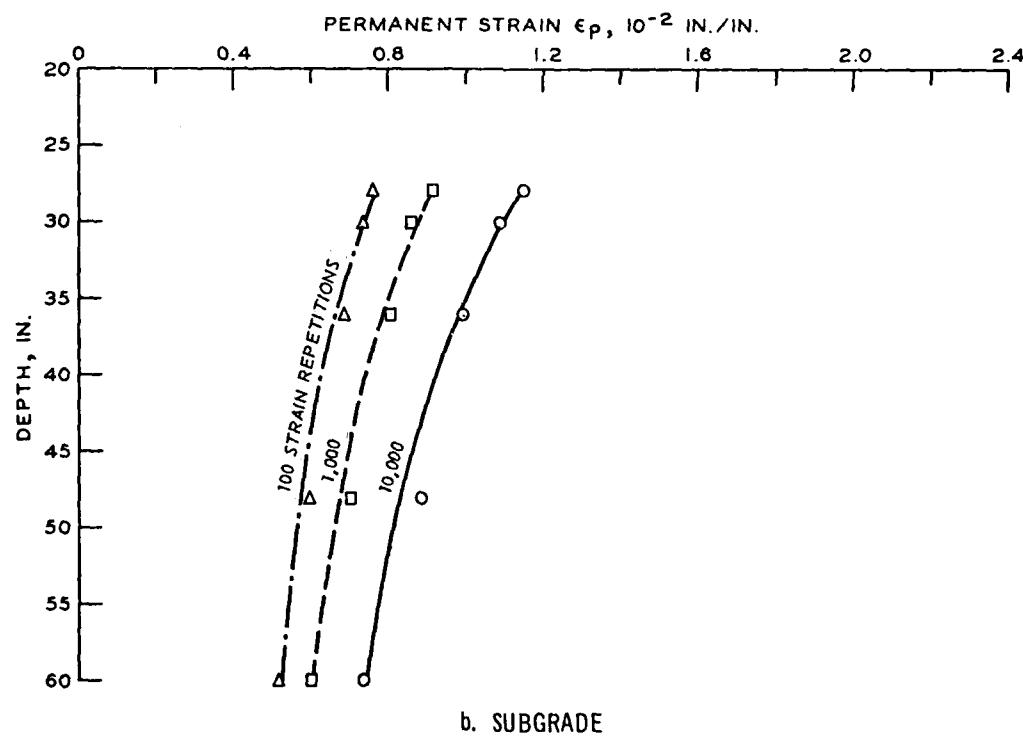
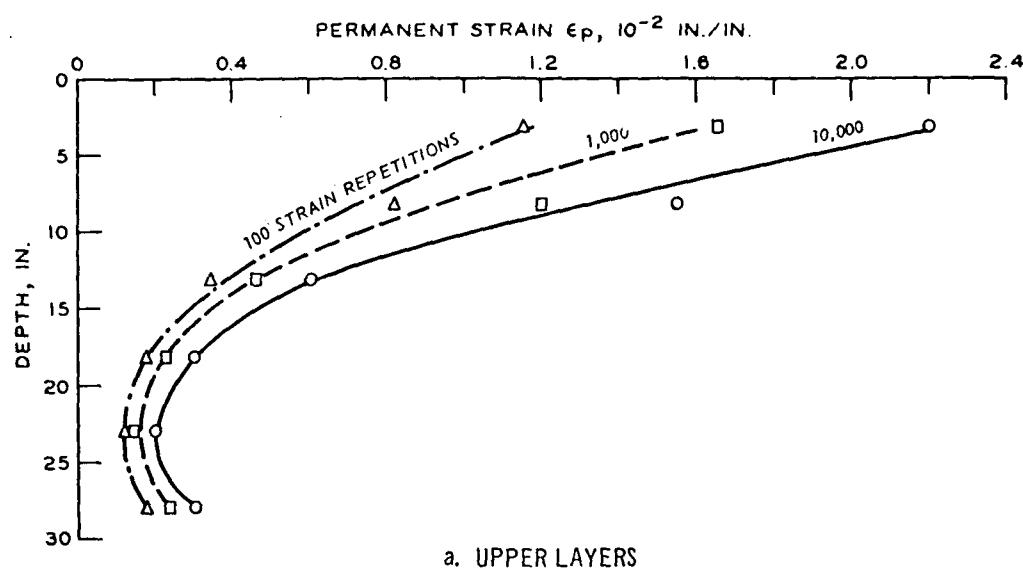


Figure B12. Computed permanent strains for pavement section 3

has also been assumed that the stabilized gravel had the same relationship between permanent and resilient strain as the crushed limestone.

To compute the total permanent deformation the function representing the distribution of the permanent strain was numerically integrated by computing the area under the distribution curves. For this computation, the permanent strain was assumed to be zero at a depth of 240 in. The total permanent deformations for the pavement sections at the different coverage levels is given in Table 5. The comparison of the computed permanent deformation with the measured permanent deformation is shown in Figure 13. For each of the test sections the permanent deformation was overestimated at 100 coverages and underestimated at failure.

There are deficiencies in the procedure, notably the assumed relationships between resilient strain and permanent strain, the inability of layered elastic theory to predict strains and the use of a coverage as a strain repetition, which affected the predicted results. Of these deficiencies probably the most serious and the most difficult to correct is the inability of the analytical model to predict the resilient strains. The accuracy of the prediction is greatly affected by the accuracy of the computed resilient strain which emphasizes the need for more accurate models for predicting responses of pavement systems.

TABLE B5
Computed Surface Deformation

<u>Pavement Section</u>	<u>@ 100 Coverages</u>	<u>@ 1000 Coverages</u>	<u>@ 10,000 Coverages</u>
1	1.452	1.785	2.344
2	0.866	1.059	1.285
3	0.772	0.933	1.143

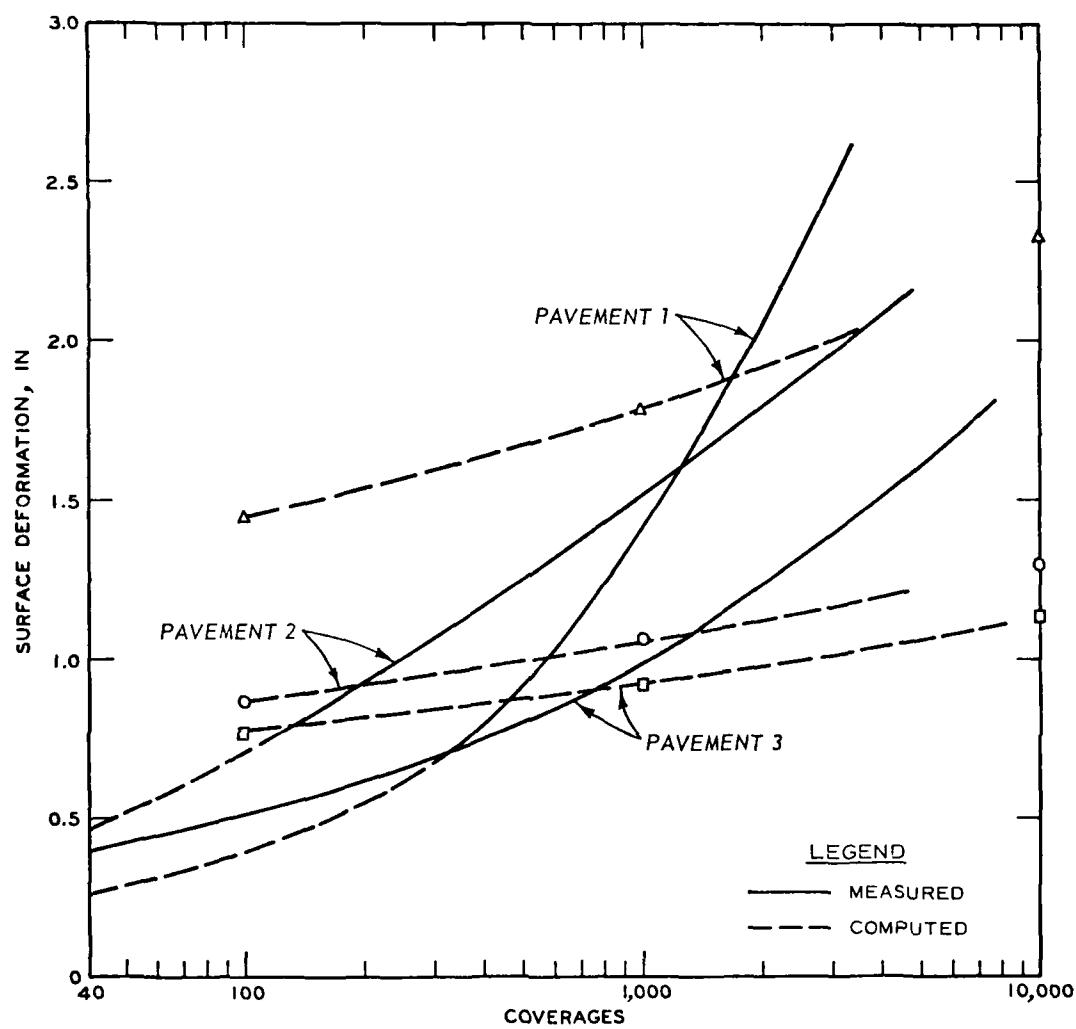


Figure B13. Comparison of computed permanent deformation with measured permanent deformation

REFERENCES

1. Brabston, W. N., Barker, W. R., and Harvey, G. G., "Development of a Structural Design Procedure for All-Bituminous Concrete Pavements for Military Roads," Technical Report (in preparation), U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
2. Barker, W. R. and Brabston, W. N., "Development of a Structural Design Procedure for Flexible Airport Pavements," Report No. FAA-RD-74- (in preparation), Federal Aviation Administration, Washington, D. C.
3. Chou, Y. T., Hutchinson, R. L., and Ulery, H. H., Jr., "A Design Method for Flexible Airfield Pavements," Transportation Research Record, No. 521, 1974, pp. 1-13.
4. Finn, F. N., Keshavan, N., and Monismith, C. L., "Applications of Theory in the Design of Asphalt Pavements," Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, Vol 1, 1972, pp. 392-409.
5. Edwards, J. M. and Valkering, C. P., "Structural Design of Asphalt Pavement for Heavy Aircraft," Shell Construction Service Report, 1971, Koninklyke-Shell/Laboratorium, Amsterdam.
6. Peattie, K. R., "A Fundamental Approach to the Design of Flexible Pavements," Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, pp. 403-411.
7. Witczak, M. W., "Design of Full-Depth Asphalt Airfield Pavements," Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, Vol 1, 1972, pp. 550-567.
8. Dorman, G. M. and Metcalf, C. T., "Design Curves for Flexible Pavements Based on Layered System Theory," Highway Research Record, No. 71, 1965, pp.
9. Barksdale, R. D., "Repeated Load Test Evaluation of Base Course Materials," GHD Research Project No. 7002, Georgia Institute of Technology, 1972.
10. Chisolm, E. E., "A Study on the Relationship of Elastic to Plastic Strain of Materials Having Different CBR Values," WESSE Memorandum report from Edward Chisolm to Walter Barker, 25 Sept 1974.
11. Ogawa, Mario, "The Permanent Deformation Characteristics of Compacted Subgrade Soil Subjected to Repeated Loading," CE 299 Report Transportation Division, Civil Engineering Department, University of California, Berkeley, Dec 1972.
12. Kalcheff, I. V. and Hicks, R. G., "A Test Procedure for Determining the Resilient Properties of Granular Materials," Journal of Testing and Evaluation, Nov 1973.

13. Fossberg, Per Egil, "Load-Deformation Characteristics of Three-Layer Pavements Containing Cement-Stabilized Base," PhD dissertation, University of California, Berkeley, Nov 1970.
14. Seed, H. B. and McNeill, R. L., "Soil Deformation Under Repeated Stress Applications," Conference on Soils for Engineering Purposes, ASTM Special Technical Publication STP No. 232, Dec 1957.
15. Brown, S. F., Lashine, A. K. F., and Hyde, A. F. L., "Repeated Load Triaxial Testing of a Silty Clay," Geotechnique, Vol XXV, Number 1, March 1975.
16. Kirwan, R. W., Glynn, T. E., and Bonner, G. A., "The Significance of Materials Properties in Flexible Pavement Analysis," Contract No. DAJA37-71-C-3066, European Research Office, United States Army, June 1973.
17. Chisolm, E. E., "Behavioral Characteristics of Gravelly Sand and Crushed Limestone for Pavement Design," Report No. FAA-RD-74, U. S. Army Engineer Waterways Experiment Station, June 1975.
18. Barker, W. R., "Structural Analysis of Insulated Layers," (in preparation), U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.
19. "Multiple-Wheel Heavy Gear Load Pavement Tests," Waterways Experiment Station, Vicksburg, Miss., Technical Report S-71-17, Vol II, Nov 1971.
20. Grau, R. W., "Evaluation of Structural Layers in Flexible Pavement," Miscellaneous Paper S-73-26, Waterways Experiment Station, Vicksburg, Miss., May 1973.
21. "Comparative Performance of Structural Layers in Pavement Systems," U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Test Report S-74-8, June 1974.

Appendix C: Program RUT

```

1000      DIMENSION CB(6+150+8+150+M+15)+10+150+TEMP+150+DD+2+15+2+
1020      ,ISTARR(2)+SSC(150)
1040      COMMON REPS,NPS,Z(150),TR,R,ZP(150)
1060      CALL ATTACH(10,"/STRESS:",3,0,1)
1080
1100      READ(10,800) LINENO,NLAY,NPS,T,P,SMEAN,SDEV,PSI
1120      *****
1140      PRINT 801,LINENO,NLAY,NPS,T,P,SMEAN,SDEV
1145      PRINT 810,PSI
1160      PRINT 802
1180      DO 10 I = 1,NLAY
1200
1220      READ(10,800) LINENO,(CB(J,I),J=1+20)
1240      *****
1260      PRINT 803,LINENO,I,(CB(J,I),J=1+20)
1280      10 CONTINUE
1300      PRINT 804
1320      DO 20 I = 1,NPS
1340      READ(10,800) LINENO,Z(I),M(I)
1360      PRINT 805,LINENO,I,M(I),Z(I)
1380      20 CONTINUE
1400
1420      1000 CONTINUE
1440      *****8190
1460
1480      READ(10,800) LINENO,REPS
1500      WRITE(6,806) REPS
1520      IF(REPS.EQ.0) GO TO 999
1540      COMPUTE ISTARR USING THE MEANS OF ALL VARIABLES
1560      TR = T/R
1580      DO 30 I = 1,NPS
1600      ZR(I) = Z(I)/R
1620      K = M(I)
1640      TEMP(I) = CB(1,K)
1660      30 CONTINUE
1680
1700      CALL STRESS(SMEAN,PSI,SS)
1720
1740      CALL DELTA(TEMP,SS,ISTARR(1),ISTARR(2))
1741      WRITE(6,808) ISTARR(1),ISTARR(2)
1760
1780      NWAR = NLAY + 1
1800      COMPUTE DD+ AND DD- FOR STRESS
1820      DUM = SMEAN + SDEV
1840
1860      CALL STRESS(DUM,PSI,SS)
1880
1900      CALL DELTA(TEMP,SS,DD(1+1,I),DD(1+1,I))
1920
1940      DUM = SMEAN - SDEV
1960

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1920      CALL STRESS (TUM-PST-SSE)
2000      CALL DELTA (TEMP-SSE-DD(2+1,1)+DD(2+1,2))
2020
2040      COMPUTE DD+ AND DD- FOR EACH LAYER
2060      DO 295 J = 1-NLAY
2080          J = J + 1
2100          JK = -1
2120          DO 295 KK = 1-2
2140              JK = JK + (-1)
2160          DO 250 L = 1-NPS
2180              K = M(L)
2200              TF(K,NE,I) GO TO 260
2220              TEMP(L) = CB(1+K)+CB(2+K)*JK
2240          GO TO 250
2260 260      TEMP(L) = CB(1+K)
2280 250  CONTINUE
2300
2320      CALL DELTA (TEMP-S-DD(KK+J,1)+DD(KK+J,2))
2340
2360 295  CONTINUE
2381      DO 350 I = 1-NVAR
2382      MPITTE(6+809) ((DD(L,I,K+L=1-2)+K=1-2))
2383 350  CONTINUE
23880      COMPUTE MEAN AND STANDARD DEVIATION
2440      DO 400 J = 1-2
2445          AVG = 1.
2446          VAR = 1.
2460      DO 395 JJ = 1-NVAR
2480          AM = (DD(1+J,I)+DD(2+J,I))/2*DSTAR(I)
2500          AVG = AVG+AM
2520          M = (DD(1+J,I)-DD(2+J,I))/2
2540          M = 1-(M/AVG)**2
2560          VAR = VAR + M
2580 395  CONTINUE
2600          AVG = AVG + DSTAR(I)
2620          VAR = DSTAR(1-VAR)
2640          DEW = AVG + VAR
2660          MPITTE(6+807) AVG+DEW+VAR
2680  *****

2700 400  CONTINUE
2720      GO TO 1000
2740 999  CONTINUE
2760 800  FORMAT /V/
2780 807  FORMAT (5X,'MEAN RUT DEPTH' =F15.5,4,
27812      5X,'DEVIATION OF RUT' =F15.5,4,
27828      5X,'VARIATION OF RUT' =F15.5)

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6730 GO TO 80
 6740 99 CONTINUE
 6750 80 CONTINUE
 6760 UPDATE ASPHALT MODULUS,
 6770 IF (YLOC < LT, MM+1) YLOC=MM+1
 6780 IF (YLOC > ST, MM-1) YLOC=MM-1
 6790 DO 70 J=1,10
 6800 IF (YLOC < LT, YLOC+10) GO TO 70
 6810 IF (YLOC > ST, YLOC-10) GO TO 70
 6820 P42 FORMAT(14.1P,2.E15,E-E15,E15,6)
 6830 MSET
 6840 GO TO 90
 6850 88 MSET-1
 6860 GO TO 90
 6870 70 CONTINUE
 7130 90 CONTINUE
 7140 NZ = 1
 7150 N1 = NZ-1
 7151 DO 400 JJ = 1,N1
 7152 DO 300 II = NZ+1
 7153 IF (ZRY(JJ),NZ,0) IT=100 GO TO 301
 7154 NZ = II
 7155 GO TO 399
 7156 301 IF (ZRY(JJ),NZ,0) IT=100 GO TO 300
 7157 TEMP = ZRY(JJ)+ZRY(IT+1)/2
 7158 IF (ZRY(JJ),NZ,TEMP) GO TO 302
 7159 ZRY(JJ) = ZRY(IT+1)
 7160 NZ = IT
 7161 Z(JJ) = ZRY(JJ)*R
 7162 GO TO 399
 7163 302 ZRY(IT) = ZRY(IT+1)
 7164 300 CONTINUE
 7165 399 CONTINUE
 7270 383 CONTINUE
 7280 XHIMEL(1)=STRATN(NZ,N9,M9)
 7290 XHIMEL(2)=STRATN(NZ,N9,M9+1)
 7300 XHIMEL(3)=STRATN(NZ,N9+1,M9)
 7310 XHIMEL(4)=STRATN(NZ,N9+1,M9+1)
 7420 HIMEL(1)=M9
 7430 HIMEL(M9)=M9
 7440
 7510 NUMFLX(M9)=M9
 7520 HIMEL(M9)=M9+1
 7530 HIMEL(M9)=M9+1
 7600 HIMEL(M9)=M9
 7610 HIMEL(M9)=M9+1
 7620 HIMEL(M9)=M9+1
 7630 DO 111 T=1,4
 7720 CC(T)=0,

```

2250      BB+T=MMEL+T
2260      TT=MMEL+T
2270      TT=MMEL+T
2280      RR+T+T*T,
2290      RR+T+T*T*T
2300      RR+T+T*T*T*T
2310      RR+T+4*RR*T*T*T*T*T*T
2320      FDFE=0
2330      FACL=FLAE(RB+BB+4+0,+4+4+CC+DET+FDFE)
2340      TEF=FDFE,BD,0+BD TO 150
2350      STOP
2360 150  RTG=CC(1)+CC(2)*XLLOC+CC(3)*YLLOC+CC(4)*XLLOC*YLLOC
2370      CC(1)=RTG + RTI
2380 400  CONTINUE
2390      TNPEN = 0,
2400      RETURN
2410
2420      SUBROUTINE FLAE(RB+BB+N1+R2+CC+DET+FDFE)
2430      JTMENCTON RB(N1+N2+BB+N1)+CC(N1)
2440      NM1=N+1
2450      DET=1,
2460
2470 150  DO 150 H=1,NM1
2480      HPI=H+1
2490      L=H
2500      DO 100 T=HPI+1
2510      TEF=RTG(T,FDFE,FLAE,RS(FLA(L+H))DO TO 100
2520      L=L
2530 100  CONTINUE
2540      TEF=LE,FDO TO 120
2550      DO 110 J=H+1
2560      TEMP=RA(J)+J
2570      RA(J+1)=RA(J)+J
2580 110  RA(J+1)=TEMP
2590      TEMP=BB(H)
2600      FPI=FPI+1
2610      FPI=FPI+1
2620      DET=DET-TEF
2630
2640 120  FTMOT=RA(FPI+1)
2650      DET=DET+FTMOT
2660      TEF=RTG(FTMOT,LE,FDO)DO TO 999
2670      DO 150 T=HPI+1
2680      FAF=RA(T+1)-FTMOT
2690      RA(T+1)=FAF
2700      DO 130 T=HPI+1
2710      RACT=RA(T+1)-FAC+RA(T)+J
2720 130  RACT=RA(T+1)-FAC+RA(T)+J
2730 150  FPT=FPT+RACT-FAC+RA(T)
2740

```

9330 CHECK LAST ELEMENT, THEN BACK TO LINE 1
9340

9340 P1WOT=RR1(N,N)
9350 DETERMINE P1WOT
9360 TEST P1WOT <= L, GO TO 939
9370 COUNT P1WOT < P1WOT
9380 T=NM1
9390 P00 TEST+1
9400 T=NM0
9410 DO P10 = P1+N
9420 P10 = NM0+P1+N-1+P1+N
9430 COUNT P10 = NM0+P1+N-1
9440 T=N-1
9450 TEST, GT, GO TO 900
9460 RETURN
9470 P00 P0DE=1
9480 RETURN
9490 END

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Cover title.
"September 1982."
Final report.
"Prepared for Assistant Secretary of the Army (R&D),
Department of the Army under Project No. 4A1G1101A910,
Task Area 02, Work Unit 15906."
Bibliography: p. 14.
1. Pavements. 2. Surface roughness. 3. United States.
Assistant Secretary of the Army (R&D). 4. U.S. Army
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